Australian Reinforced Concrete Design Handbook

Cement and Concrete Association of Australia

Australian Reinforced Concrete Design Handbook

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The Cement and Concrete Association of Australia is a national, non-profit organisation, sponsored by the cement industry to provide information on the many uses of cement and concrete. This publication has been prepared for general information and in furtherance of the Association's objects only. Since it is not intended to replace the services of professional consultants on particular projects, no legal liability can be accepted by the Association for its use.

Australian Reinforced Concrete Design Handbook

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Cement and Concrete Association of Australia
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Association of Consulting Structural Engineers of N.S.W.

Foreword

Designers who follow the requirements of the Australian Code for Reinforced Concrete (AS CA2) have long been handicapped by the absence of a design handbook based specifically on this code.

This handbook has been produced to meet this need. It deals with reinforced concrete design based on permissible stresses, covering flexural members, compression members and footings. Notation accords with AS CA2 as do dimensions except for moments which are expressed in in. kips and flexural spans which are in feet.

A section of the handbook deals with derivation of design formulae and is intended to be of use primarily to students of reinforced concrete design as well as to their tutors. For the new or infrequently practised designer, the design examples showing the application of the tables will provide guidance. It is hoped that the experienced designer also will find the use of these design aids straightforward, and that they will alleviate some of the design effort hitherto required when designing in accordance with AS CA2.

The efforts of many people are required in the production of a handbook such as this. The Association is particularly indebted to the C.S.I.R.O. Division of Building Research for making available computer facilities, and for the programming of the design formulae so that the computer print-out followed the designated table format. It is interesting to note that the column design diagrams (Diagrams 21, 22 & 23) were initially drawn by the computer to designated scale. In particular, the assistance of Dr. F. A. Blakey, Dr. J. F. Brotchie and Mr. F. D. Beresford is acknowledged.

The Association is also indebted to a sub-committee of the Association of Consulting Structural Engineers of N.S.W. who advised on and reviewed the work as it progressed. This sub-committee consisted of Convenor — Mr. M. G. Bull; Mr. R. F. Chapman, Mr. J. E. Delprado, Mr. I. Morrison and Mr. A. J. Taylor.

Despite the work which goes into their production, design tables and charts tend to become obsolete as new codes of practice are prepared. To provide for this, the Association proposes to issue new tables and charts as these become necessary. Users of the handbook are invited, therefore, to register with the Association so that they may be advised of the availability of amendments or new sections as these are prepared.

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NOTATION

Symbol	Definition and Unit
a	coefficient $= \frac{F_s j}{1{,}000}$; used in $A_{st} = \frac{M}{ad}$ and in $A_{st} = \frac{Ne}{adi}$
a_f	same as 'a' but for flange only of T and L—sections
a_w	same as 'a' but for web only of T and L—sections
A_b	equivalent area of helical reinforcement, computed as the volume of steel in the helix per unit length of the compression member (sq. in.)
A_g	gross cross-sectional area of a compression member, excluding any surface finishing material (sq. in.)
A_k	cross-sectional area of the core of a helically reinforced compression member, measured to the outside diameter of the helix (sq. in.)
A_s	cross-sectional area of longitudinal reinforcement (sq. in.)
A_{sc}	cross-sectional area of compressive reinforcement (sq. in.)
A_{st}	cross-sectional area of tensile reinforcement (sq. in.)
A_{sf}	that part of tensile reinforcement in T and L—sections required to balance compression in the flange (sq. in.)
A_{sw}	that part of tensile reinforcement in T and L—sections required to balance compression in the web (sq. in.)
A_{sv}	cross-sectional area of shear reinforcement per spacing interval s (sq. in.)
b	effective width at the compression face of a member (in.)
b_w	effective width of the web of T and L—sections (in.)
B	width of footing (in.)
B	$\sin\theta + \cos\theta$, as applied to shear reinforcement
c	coefficient used in $A_{sc} = \frac{M - KF}{cd}$ and in $A_{sc} = \frac{Ne - KF}{cd}$
c	equivalent column side dimension as used in design of reinforced square spread footings (in.)
c_o	concrete cover to reinforcement (in.)
C	resultant of compressive forces in flexural computations (lbf.)
d	distance from extreme compressive fibre to the centroid of tensile reinforcement (in.)
d′	distance from extreme compressive fibre to the centroid of compressive reinforcement (in.)
d″	distance from the centreline of concrete cross-section to the centroid of tensile reinforcement (in.)
D	overall depth of a flexural member or footing (in.)
D	overall diameter of a round compression member (in.)
D	overall depth of a rectangular compression member (in.)
e	eccentricity of axial load N, measured from the centroid of tensile reinforcement in a flexural member (in.)
e′	eccentricity of axial load N, measured from the centroidal axis of a column section (in.)
E_c	modulus of elasticity of concrete (psi.)
E_s	modulus of elasticity of steel reinforcement (psi.)
f_c	compressive stress in extreme fibre of concrete (psi.)
f_s	tensile stress in reinforcement (psi.)

f_{sc} compressive stress in reinforcement (psi.)

f_v maximum shear stress across a reinforced concrete section (psi.)

f'_v shear stress taken by shear reinforcement (psi.)

F_c permissible compressive stress in concrete (psi.)

F'_c minimum compressive strength of concrete required by the designer at 28 days (psi.)

F_{ct} permissible flexural tensile stress in concrete (psi.)

F $\frac{bd^2}{1{,}000}$; used in determination of resisting moment (M_R) of concrete sections

F_f same as 'F' but for flange only of T and L–sections

F_w same as 'F' but for web only of T and L–sections

F_s permissible tensile stress in longitudinal steel reinforcement (psi.)

F_{sc} permissible compressive stress in steel reinforcement (psi.)

F_{sv} permissible tensile stress in shear reinforcement (psi.)

F_v permissible average shear stress across a concrete section (psi.)

g ratio of diameter of circle (gD) through bar centrelines or distance (gD) between bars at opposite faces of a column, to overall dimension (D)

h effective length of a compression member (in.)

H_a tensile stress which can be developed in a reinforcing bar by a standard hook (psi.)

i $\frac{1}{1-\frac{id}{e}}$; used in design of sections subject to combined bending and axial load

i_f same as 'i' but for flange only of T and L–sections

i_w same as 'i' but for web only of T and L–sections

I moment of inertia of column section about axis of bending ($in.^4$)

j ratio of distance (jd) between resultants of compressive and tensile forces, to effective depth (d)

j_f same as 'j' but for flange only of T and L–sections

j_w same as 'j' but for web only of T and L–sections

k ratio of distance (kd) between neutral axis and extreme fibre, to effective depth (d) for flexural members

k ratio of distance (kD) between neutral axis and extreme fibre, to overall dimension (D) for compression members

K $\frac{1}{2} F_c k j$ for flexural members

K_f same as 'K' but for flange only of T and L–sections

K_w same as 'K' but for web only of T and L–sections

l_a embedded or anchorage length of reinforcement in concrete (in.)

L effective span of a flexural member

m $npi + (2n-1)p' + p''$; used in determination of k

M total applied bending moment (in. kips)

M_R resisting moment of concrete stresses (in. kips)

n modular ratio $= \frac{E_s}{E_c}$

n′ effective modular ratio as used in investigation of doubly reinforced flexural members

N total applied eccentric load (kips)

p bearing pressure under a footing (kips per sq. ft.)

p ratio of area of tensile reinforcement in flexural members or total reinforcement in compression members, to effective area of concrete at a cross-section

p' ratio of area of compressive reinforcement in flexural members to effective area of concrete

p'' $\frac{(b - b_w)t}{b_w d}$ for T and L—sections

P applied coaxial load to a compression member (kips)

q $npi + (2n - 1)p' \frac{d'}{d} + p'' \frac{t}{2d}$; used in determination of k

r radius of gyration of column section about axis of bending (in.)

s centre to centre distance between reinforcing bars (in.)

s spacing of shear reinforcement measured parallel to neutral axis (in.)

s pitch of helical reinforcement (in.)

t flange thickness of a T or L—section

T resultant of tensile forces in flexural computations (lbf.)

u calculated local bond stress (psi.)

u_{av} calculated average bond stress (psi.)

U permissible local bond stress (psi.)

U_{av} permissible average bond stress (psi.)

V gross shear force acting across a section (kips)

V' shear force resisted by shear reinforcement (kips)

w total distributed load per unit length or area (lbf. per ft. or lbf. per sq. ft.)

w_D total distributed dead load per unit length or area (lbf. per ft. or lbf. per sq. ft.)

w_L total distributed live load per unit length or area (lbf. per ft. or lbf. per sq. ft.)

W applied concentrated load to a flexural member (lbf.)

x outstand or cantilever length of a footing as measured from column face to edge of footing (in.)

y ratio of distance (yt) between extreme fibre and resultant of compressive forces, to flange thickness (t)

z ratio of distance (zkd) between extreme fibre and resultant of compressive forces, to depth (kd)

β coefficient $= \sqrt{\frac{20.83\ p}{F_{ct}}}$; used in $D = \beta x$ in design of plain footings

θ angle between axis of shear reinforcement and neutral axis

Σo sum of perimeters of tensile reinforcing bars (in.)

ϕ, ϕ_1, ϕ_2 diameter of reinforcing bars (in.)

ϕ plain, structural grade, round section reinforcing bar (followed by appropriate size number)

\# deformed, round section reinforcing bar (followed by appropriate size number)*

---

* The symbol # is used in practice to denote only hard grade deformed reinforcement. Structural grade, and cold worked grade deformed reinforcement are denoted respectively by #S and #C, followed by an appropriate size number.

Derivation of Design Formulae

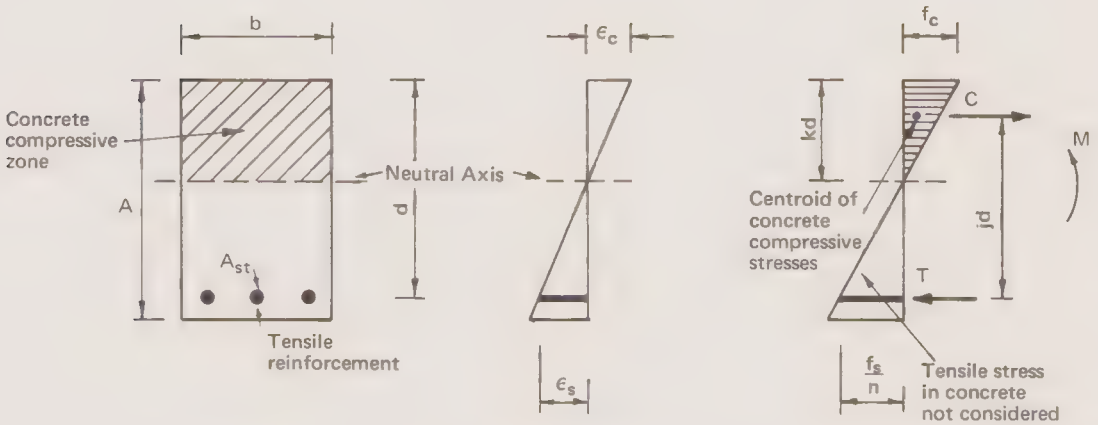
In this section, the derivation has been given of most of the equations used in the preparation of tables or diagrams during the solution of examples. Of necessity, the derivations are restricted almost entirely to the presentation of the algebraic relationships of simple elastic behaviour. For discussion in detail of the basis for, and the advantages and disadvantages in practice of the elastic approach, the reader is referred to the many standard texts that are available.

In referring to stresses in the handbook, F with an appropriate subscript has been used for permissible values laid down in AS CA2–1963, and f for calculated actual stresses that are not necessarily equal to permissible values. Thus in design examples and associated tables, the relevant equations are normally expressed in terms of the initially specified permissible stresses F, while in investigation examples f is used since stresses are the unknown quantities to be determined. In the derivations which follow however, design and investigation equations are closely inter-related. Consequently, for simplicity, most of the equations have been derived in terms of the general stress f, and design equations have then been rewritten where necessary in terms of permissible stresses.

A DESIGN OF FLEXURAL MEMBERS

A1 Rectangular Section

A1–1 Section Coefficients



(a) Cross-section (b) Strain distribution (c) Stress distribution

Fig. A1

The basic relationships of the elastic behaviour of reinforced concrete may be derived from a consideration of the distribution of strain and stress in a rectangular cross-section as shown in Fig. A1.

Assume (i) plane cross sections remain plane during bending (Fig. A1 (b)), and
(ii) tensile stresses in concrete are ignored.

Denote strain in extreme concrete compressive fibre as ϵ_c, average strain in tensile reinforcement as ϵ_s, modulus of elasticity for concrete as E_c, and modulus of elasticity for steel as E_s.

Then $\epsilon_c = \frac{f_c}{E_c}$, $\epsilon_s = \frac{f_s}{E_s}$ and by definition $n = \frac{E_s}{E_c}$,

so that $\frac{\epsilon_c}{\epsilon_s} = \frac{f_c}{f_s/n}$.

It follows that a linear stress diagram may be drawn if, corresponding to a maximum concrete compressive stress f_c, the steel stress is drawn as $\frac{f_s}{n}$ (Fig. A1 (c)).

The resultant force C of the triangular distribution of concrete compressive stresses is given by

$$C = \frac{f_c\, k\, d\, b}{2},$$

and the resultant steel tensile force T by

$$T = f_s\, A_{st} = f_s\, pbd,$$

where $p = \frac{A_{st}}{bd}$, by definition.

(1) $K = \frac{M_R}{bd^2}$ (by definition) $= \frac{f_c\, k\, j}{2} = f_s\, p\, j,$

where M_R is the moment of resistance of concrete compressive stresses.

In terms of permissible stresses,

$K = \frac{F_c\, k\, j}{2} = F_s\, p\, j.$ See Table 1.

Taking moments about T, $M_R = C \times jd,$

so that $Kbd^2 = \frac{f_c\, k\, d\, b}{2} \times jd$

and $K = \frac{f_c\, k\, j}{2}.$

Taking moments about C, $M_R = T \times jd,$

so that $Kbd^2 = f_s\, pbd \times jd$

and $K = f_s\, p\, j.$

(2) $k = \frac{1}{1 + \frac{f_s}{nf_c}}.$

In terms of permissible stresses,

$k = \frac{1}{1 + \frac{F_s}{nF_c}}.$ See Table 1.

From Fig. A1 (c), by similar triangles,

$$\frac{kd}{d - kd} = \frac{f_c}{f_s/n},$$

from which Equation (2) follows.

Note that Equation (2) applies to all cross-sectional shapes.

(3) $j = 1 - \frac{k}{3}.$ See Table 1.

For a triangular distribution of concrete compressive stresses, and no compressive reinforcement, C is located a distance $\frac{kd}{3}$ from the extreme fibre, so that

$$d = jd + \frac{kd}{3},$$

from which Equation (3) follows.

(4) $A_{st} = \frac{M\ (\text{in. lb.})}{f_s\, jd} = \frac{1{,}000\ M\ (\text{in. kips})}{f_s\, jd}$

In terms of permissible stresses,

$A_{st} = \frac{1{,}000\ M\ (\text{in. kips})}{F_s\, jd} = \frac{M\ (\text{in. kips})}{ad}$, where M is the externally applied bending moment, and

$a = \frac{F_s j}{1{,}000}$, by definition. See Table 1.

Equation (4), which applies to simple bending, follows directly from the derivation of Equation (1).

(5) $A_{st} = \frac{Ne}{adi}$ for combined bending and axial load,

where N = external load actually applied at mid-depth of the concrete cross-section, or equivalent eccentric load acting at a distance e from T,

and $i = \frac{1}{1 - \frac{jd}{e}}$. See Table 9.

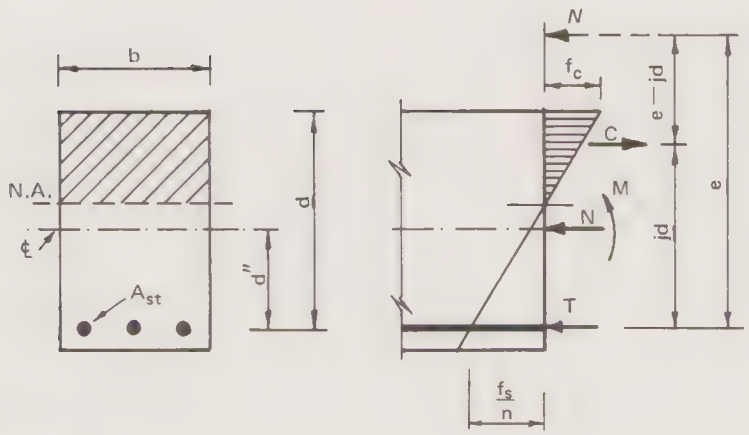


Fig. A2

To determine e : The moment about T of M and axial load N = M + Nd″, and the moment about T of the equivalent eccentric load N = Ne.

Therefore Ne = M + Nd″

or $e = \frac{M}{N} + d''$.

To derive Equation (5) : Taking moments about C,

$N(e - jd) - Tjd = 0$

$= N(e - jd) - \frac{A_{st}\, f_s\, jd}{1{,}000}$.

Therefore $A_{st} = N(e - jd) \times \frac{1{,}000}{f_s\, jd}$

$= \frac{1{,}000Ne}{f_s\, jd}\left(1 - \frac{jd}{e}\right)$.

Writing F_s for f_s, $i = \frac{1}{1 - \frac{jd}{e}}$ and $a = \frac{F_s j}{1{,}000}$, $A_{st} = \frac{Ne}{adi}$.

Equation (5) applies also to simple bending, in which case e = infinity, Ne = M and i = unity. It does not apply to those combinations of bending moment and relatively large axial load for which the equivalent eccentric load N lies between C and T, and which should be treated as eccentrically loaded compression members. The latter restriction on the location of the equivalent load N follows from equilibrium requirements. For compressive N acting between C and T, T must change to compression also, while for N tensile, the compression concrete is required to support tension; both conditions are inadmissible.

(6) $$p = \frac{A_{st}}{bd} = \frac{1}{\frac{2f_s}{f_c}\left(1 + \frac{f_s}{nf_c}\right)} = \frac{f_c k}{2f_s}$$ (simple bending).

In terms of permissible stresses,

$$p = \frac{1}{\frac{2F_s}{F_c}\left(1 + \frac{F_s}{nF_c}\right)} = \frac{F_c k}{2F_s}.$$ See Table 1.

(7) $$p = \frac{f_c k}{2f_s i}$$ (combined bending and axial load).

In terms of permissible stresses,

$$p = \frac{F_c k}{2F_s i}.$$

In Equations (6) and (7), p, when expressed in terms of the permissible stresses F_s and F_c, is referred to as the balanced steel ratio, and the corresponding value of A_{st} is the balanced steel area.

To derive Equation.(7), $\frac{Ne}{adi}$ is substituted for A_{st} in the equation $p = \frac{A_{st}}{bd}$:

$$p = \frac{Ne}{abd^2 i}$$

$$= \frac{1{,}000\,Ne}{F_s j b d^2 i}$$

$$= \frac{M_R}{F_s j b d^2 i}$$

From Equation (1) $M_R = Kbd^2$, and $K = \frac{F_c k j}{2}$.

Therefore

$$p = \frac{Kbd^2}{F_s j b d^2 i}$$

$$= \frac{F_c k j}{2F_s j i}$$

$$= \frac{F_c k}{2F_s i}$$ (balanced p).

A1–3 Resisting Moment of Concrete Stresses

(8) $M_R = KF$ (in. kips), where

$F = \frac{bd^2}{1{,}000}$, by definition. See Table 4.

From Equation (1), for rectangular section,

$$K = \frac{f_c\, kj}{2}.$$

Taking moments of concrete compressive stresses about T,

$$\begin{aligned} M_R &= \tfrac{1}{2} f_c\, b\, kd\, jd \\ &= \tfrac{1}{2} f_c\, kj\, bd^2 \\ &= K\, bd^2 \text{ (in. lb.)} \\ &= K \frac{bd^2}{1{,}000} \text{ (in. kips).} \end{aligned}$$

A1–4 Compressive Reinforcement

Compressive reinforcement is required whenever the externally applied moment M or Ne exceeds the moment of resistance of concrete compressive stresses KF.

(9) $A_{sc} = \frac{M - KF}{cd}$ (simple bending).

(10) $A_{sc} = \frac{Ne - KF}{cd}$ (combined bending and axial load).

For (i) $k \leq \frac{F_{sc} + 2F_s \frac{d'}{d}}{F_{sc} + 2F_s}$,

$$c = \frac{F_s}{1{,}000} \times \frac{2n-1}{n} \times \frac{(1-\frac{d'}{d})(k-\frac{d'}{d})}{(1-k)}.$$ See Table 7.

For (ii) $k > \frac{F_{sc} + 2F_s \frac{d'}{d}}{F_{sc} + 2F_s}$,

$$c = \frac{F_{sc}}{1{,}000}\left[1 - \frac{F_c}{F_{sc}}\left(\frac{kd - d'}{kd}\right)\right](1-\frac{d'}{d}).$$ See Table 7.

Note that the coefficient c is given in terms of permissible stresses.

The value of $k = \frac{F_{sc} + 2F_s \frac{d'}{d}}{F_{sc} + 2F_s}$ is referred to as the transition value.

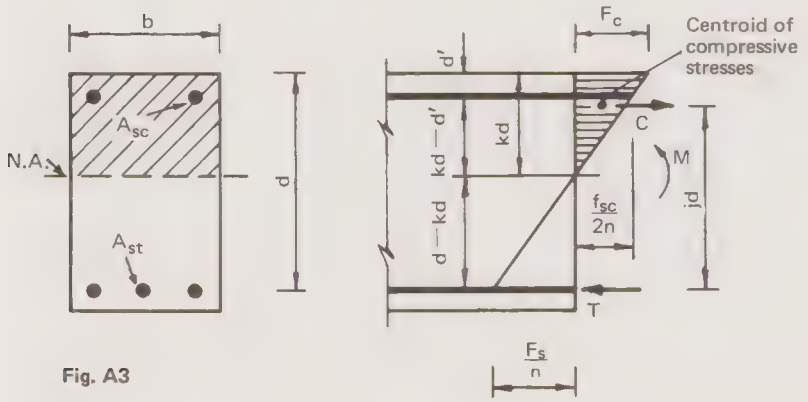


Fig. A3

(i) The balanced rectangular section in Fig. A3 is subject to bending moment M which results in concrete and tensile reinforcement stresses F_c and F_s respectively. The stress in the compressive reinforcement, transformed at a modular ratio of 2n, is f_{sc}.

$$\text{From similar triangles } \frac{\frac{f_{sc}}{2n}}{kd-d'} = \frac{\frac{F_s}{n}}{d-kd},$$

$$\text{so that } \frac{f_{sc}}{2n}(d-kd) = \frac{F_s}{n}(kd-d').$$

Multiply both sides by $\frac{2n}{d}$, and rearrange to give

$$k = \frac{f_{sc}+2F_s\frac{d'}{d}}{f_{sc}+2F_s}.$$

It follows that for $f_{sc} \leqslant F_{sc}$,

$$k \leqslant \frac{F_{sc}+2F_s\frac{d'}{d}}{F_{sc}+2F_s}.$$

To determine c, take moments about tensile reinforcement.

$$\text{Moment of compressive reinforcement} = f_{sc}A_{sc}(d-d').$$

$$\text{Moment of concrete stresses} = \tfrac{1}{2}F_c bkd\,jd - \frac{f_{sc}}{2n}A_{sc}(d-d').$$

The term $\frac{f_{sc}}{2n}A_{sc} \times (d-d')$ is the moment of an area of concrete A_{sc}, in which the stress would have been $\frac{f_{sc}}{2n}$, which is displaced by the compressive reinforcement.

Then total moment about T is

$$M \text{ (in. lb.)} = \tfrac{1}{2}F_c bkdjd + \frac{(2n-1)}{2n}f_{sc}A_{sc}(d-d'),$$

$$\text{or } M \text{ (in. kips)} = KF + \frac{(2n-1)}{2n} \times \frac{f_{sc}A_{sc}}{1{,}000}(d-d').$$

From the similar triangles relationship above,

$$f_{sc} = 2F_s\frac{(k-\frac{d'}{d})}{(1-k)}.$$

Therefore

$$M \text{ (in. kips)} = KF + \frac{(2n-1)}{2n} \times \frac{2F_sA_{sc}}{1{,}000} \times \frac{(k-\frac{d'}{d})}{(1-k)} \times (d'-d)$$

$$= KF + \frac{F_sA_{sc}}{1{,}000} \times \frac{(2n-1)}{n} \times \frac{(1-\frac{d'}{d})\,d\,(k-\frac{d'}{d})}{(1-k)}.$$

$$\text{Let } c = \frac{F_s}{1{,}000} \times \frac{(2n-1)}{n} \times \frac{(1-\frac{d'}{d})(k-\frac{d'}{d})}{(1-k)}, \text{ and}$$

solve for A_{sc} to give Equation (9).

(ii) For $k > \frac{F_{sc}+2F_s\frac{d'}{d}}{F_{sc}+2F_s}$, it follows from (i) above that $f_{sc} > F_{sc}$ when a modular ratio of 2n is used to transform the compressive reinforcement. This inequality is not acceptable to the designer, who cannot assume a working stress greater than that permitted by the code.

There are two courses open to the designer. One course is to reduce the actual value of k for the section in question until it is equal to the transition value. From Equation (2) this step can be achieved by reducing the concrete compressive stress, which in turn may lead to a considerable reduction in the resultant concrete compressive force. Consequently a large amount of compressive reinforcement, at stress F_{sc}, may be required. The alternative procedure, which is adopted in this handbook, is to provide sufficient compressive reinforcement, at assumed stress F_{sc}, to balance the excess moment (M — KF) or (Ne — KF). In this instance, the concrete and tensile steel stresses F_c and F_s are maintained, so that in effect the compressive reinforcement is being transformed at some particular modular ratio less than 2n. For design purposes it is not necessary to know the value of the reduced modular ratio, but in investigation examples, it may be computed.

The second expression for coefficient c is then derived as follows:

Taking moments of compressive forces about the tensile reinforcement,

$$M \text{ (in. lb.)} = \tfrac{1}{2} F_c b\, kd\, jd + F_{sc} A_{sc} (d - d') - F_c \left(\frac{kd - d'}{kd}\right) A_{sc} (d - d').$$

The last term of this equation is the moment of the force in the area of concrete A_{sc} which is displaced by the compressive reinforcement.

Writing $$K = \tfrac{1}{2} F_c kj \text{ and } F = \frac{bd^2}{1{,}000},$$

$$M \text{ (in. kips)} = KF + \frac{F_{sc} A_{sc}}{1{,}000} \times \left(1 - \frac{d'}{d}\right) d - \frac{F_c A_{sc}}{1{,}000} \times \left(\frac{kd - d'}{kd}\right) \times \left(1 - \frac{d'}{d}\right) d$$

$$= KF + A_{sc}\, d \times \frac{F_{sc}}{1{,}000} \left[1 - \frac{F_c}{F_{sc}} \left(\frac{kd - d'}{kd}\right)\right] \left(1 - \frac{d'}{d}\right),$$

so that $$c = \frac{F_{sc}}{1{,}000} \left[1 - \frac{F_c}{F_{sc}} \left(\frac{kd - d'}{kd}\right)\right] \left(1 - \frac{d'}{d}\right).$$

The computation of compressive reinforcement for sections subject to combined bending and axial load is the same as for simple bending except that the external moment M is replaced by Ne. Values of c in Table 7 apply to both types of loading.

A1–5 Tensile Reinforcement in Sections Having Compressive Reinforcement

It is generally sufficiently accurate to use Equations (4) or (5) to determine A_{st} for sections containing compressive reinforcement, taking the value of a or j from Table 1. For deep beams with heavy compressive reinforcement however, greater accuracy may be obtained from the following equations (written in terms of permissible stresses).

(11) $$A_{st} = pbd + \frac{1{,}000\,(M - KF)}{F_s\,(d - d')} \quad \text{(simple bending)};$$

(12) $$A_{st} = \frac{pbd}{i} + \frac{1{,}000\,(Ne - KF)}{F_s\,(d - d')\,i} \quad \text{(combined bending and axial load)},$$

where p is the balanced steel ratio as given in Table 1.

In each equation the first term is the area of reinforcement required to balance the concrete compressive stresses, the moment of which is KF. The second term is the tensile reinforcement area to balance the excess moment (M − KF) or (Ne − KF). The difference between A_{st} computed from Equation (4) or (5), and (11) or (12) is evidently greatest when this second term is large, and when (d − d′) does not approximate closely to jd.

A2 T–Section with Compressive Stress in Stem Neglected

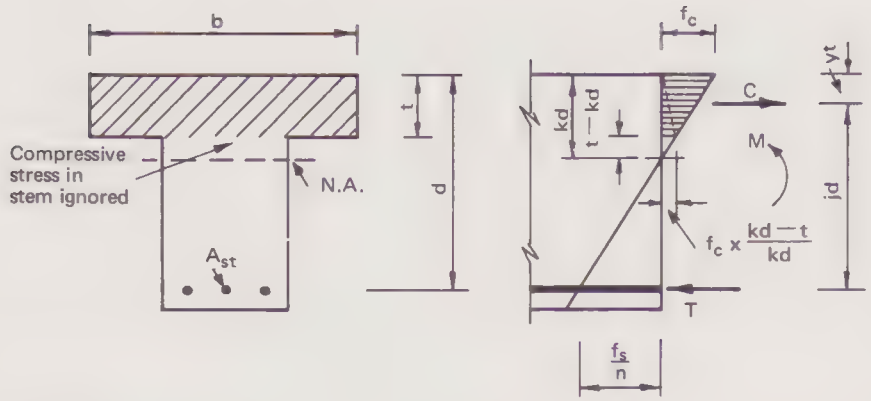


Fig. A4

A2–1 Section Coefficients

(13) $$y = \frac{1-\frac{2t}{3kd}}{2-\frac{t}{kd}}.$$ See Table 8.

The diagram of compressive stresses is a trapezoid, the centroid of which can be located from the usual equation. The stress at the underside of the flange must first be established; from similar triangles it is $f_c \times \frac{kd-t}{kd} = f_c\,(1-\frac{t}{kd})$.

Then $$yt = \frac{t}{3} \times \frac{f_c + 2f_c\,(1-\frac{t}{kd})}{f_c + f_c\,(1-\frac{t}{kd})},$$

and $$y = \tfrac{1}{3} \times \frac{1+2-\frac{2t}{kd}}{1+1-\frac{t}{kd}}$$

$$= \frac{1-\frac{2t}{3kd}}{2-\frac{t}{kd}}.$$

(14) $$j = 1-\frac{yt}{d} = 1-\frac{k-\frac{2t}{3d}}{2k-\frac{t}{d}} \times \frac{t}{d}.$$ See Table 8.

Since jd is the lever arm between the tensile reinforcement and the resultant C of the concrete forces,

$$jd = d - yt,$$

$$\text{so that } j = 1-\frac{yt}{d}.$$

Substitute for y, using Equation (13), to obtain Equation (14).

A2–2 Resisting Moment of Concrete Stresses

The coefficient $F = \frac{bd^2}{1{,}000}$ remains the same as for the rectangular section (see Table 4).

Then

$M_R = K\frac{bd^2}{1{,}000} = KF$ (in. kips), in which

(15) $K = \frac{f_c}{2} \times \frac{t}{d}\left(2 - \frac{t}{d} - \frac{t}{kd} - \frac{2t^2}{3kd^2}\right)$.

In terms of permissible stresses,

$K = \frac{F_c}{2} \times \frac{t}{d}\left(2 - \frac{t}{d} - \frac{t}{kd} - \frac{2t^2}{3kd^2}\right)$. See Table 8.

Equation (15) can be derived most conveniently by subdividing the compressive stress diagram (Fig. A4) into two triangular sections, and taking moments of each triangle about the tensile reinforcement.

$$M_R = \tfrac{1}{2} f_c bt\left(d - \tfrac{t}{3}\right) + \tfrac{1}{2} f_c \left(1 - \frac{t}{kd}\right) bt\left(d - \frac{2t}{3}\right)$$

$$= \tfrac{1}{2} f_c\, bt\left(2d - t - \frac{t}{k} + \frac{2t^2}{3kd}\right).$$

Divide by bd^2 and put $\frac{M_R}{bd^2} = K$:

$$K = \frac{f_c}{2} \times \frac{t}{d}\left(2 - \frac{t}{d} - \frac{t}{kd} - \frac{2t^2}{3kd^2}\right).$$

Note that Equations (13), (14) and (15) are valid only if $t \leqslant kd$. For $t > kd$, the section becomes effectively rectangular, and the equations of Section A1 apply.

A2–3 Areas of Reinforcement

For tensile reinforcement A_{st}, Equations (4) and (5) are used in the form:

$A_{st} = \frac{M \text{ (in. lb.)}}{F_s jd} = \frac{1{,}000\, M \text{ (in. kips)}}{F_s jd}$ (simple bending), and

$A_{st} = \frac{Ne \text{ (in. lb.)}}{F_s jdi} = \frac{1{,}000\, Ne \text{ (in. kips)}}{F_s jdi}$ (combined bending and axial load).

Values of j are given in Table 8, and of i in Table 9.

For compressive reinforcement, Equations (9) and (10) are used, with values of c taken from Table 7.

A3 T–Section with Compressive Stress in Stem Included

It is possible to derive from first principles, design equations for T–sections in which allowance is made for compressive stresses acting in that part of the stem lying between the flange and the neutral axis. However it is simpler and sufficiently accurate, to consider the section to be composed of two parts, namely (i) a T–section of width $(b - b_w)$, in which stresses in the stem are ignored, and (ii) a rectangular section of width b_w (see Fig. A5).

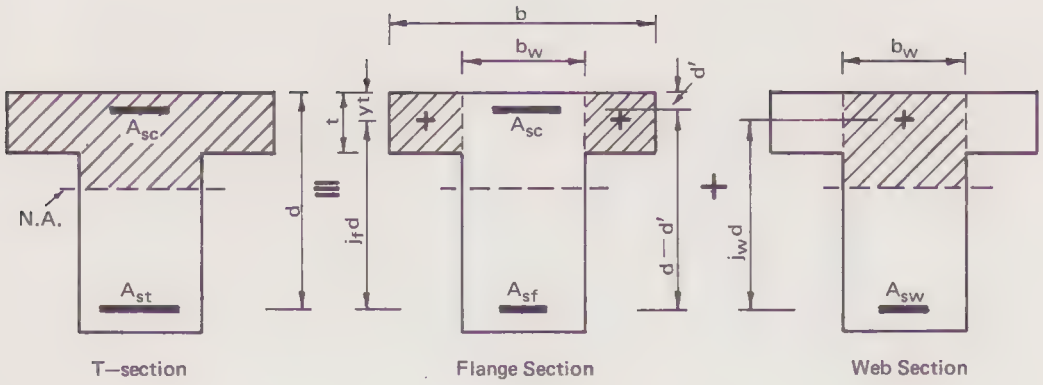


Fig. A5

A3–1 Resisting Moment of Concrete Stresses

The resisting moment of concrete stresses is the sum of the resisting moments of the web and the flange.

(16) $M_R = K_w F_w + K_f F_f = \Sigma KF$ (in. kips),

where K_w = K from Equation (1) and Table 1;

K_f = K from Equation (15) and Table 8;

$F_w = \frac{b_w d^2}{1{,}000}$

$F_f = \frac{(b - b_w)d^2}{1{,}000}$

} From Table 4.

A3–2 Areas of Reinforcement

(17) $A_{sc} = \frac{M - \Sigma KF}{cd}$ (simple bending).

(18) $A_{sc} = \frac{Ne - \Sigma KF}{cd}$ (combined bending and axial load).

Values of c are taken from Table 7.

Tensile reinforcement may be computed as follows:

$A_{st} = A_{sw} + A_{sf}$, where

(19) $A_{sw} = \frac{K_w F_w}{a_w d i} = \frac{M_R}{a_w d i} = p_w b_w d$ (see Equations (4), (5) and (8)).

For simple bending, i = unity.

(20) $A_{sf} = \frac{M - K_w F_w}{F_s j_f d}$ (simple bending).

(21) $A_{sf} = \frac{Ne - K_w F_w}{F_s j_f d i}$ (combined bending and axial load).

In Equation (19), a_w is taken from Table 1, and in Equations (20) and (21), j_f is taken from Table 8.

Since $K_w F_w$ is the resisting moment of concrete stresses for the rectangular section, A_{sw} is the balanced area of reinforcement $= p_w b_w d$. A_{sf} is then the additional area of reinforcement required to balance the excess moment $(M - K_w F_w)$ or $(Ne - K_w F_w)$. Note that if M or Ne is less than $K_w F_w$, A_{sf} is negative and must be subtracted from A_{sw} to give the nett area of reinforcement A_{st}. In this circumstance, as an alternative, the section can be treated as simply rectangular and A_{st} computed directly from Equation (4) or (5).

A4 Bond Stresses

A4–1 Local bond

(22) $u = \frac{V}{\Sigma_o j d}$.

To derive Equation (22), consider the reinforced beam in Fig. A6 (a). A portion of length Δx cut from this beam, together with the forces acting on it, is shown in Fig. A6(b).

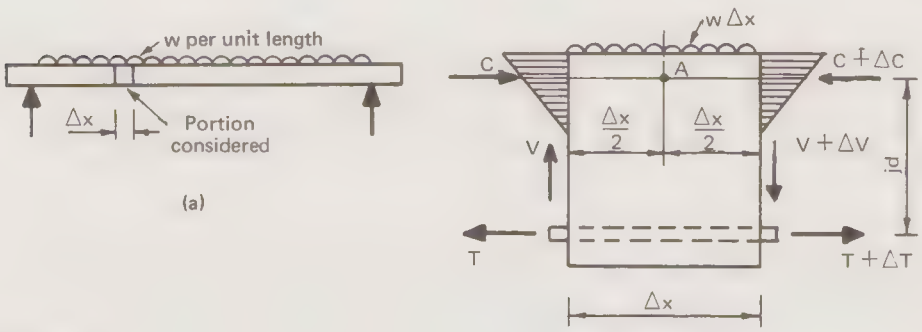


Fig. A6

Taking moments about A,

$$Tjd + V\frac{\Delta x}{2} + (V + \Delta V)\frac{\Delta x}{2} - (T + \Delta T)jd = 0.$$

Ignoring second order small quantities, this simplifies to:

$$\frac{\Delta T}{\Delta x} = \frac{V}{jd}.$$

ΔT is the change in force in the tensile reinforcement over the length Δx, which must be resisted by bond between the reinforcement and concrete.

Then for a perimeter length Σo

$$\Delta T = u\,\Sigma o\,\Delta x \text{ or } \frac{\Delta T}{\Delta x} = u\Sigma o.$$

Therefore $u\Sigma o = \frac{V}{jd}$ or

$$u = \frac{V}{\Sigma_o j d}.$$

The permissible local bond stress U in effect limits the rate of change of tensile force or stress per unit length of reinforcement.

A4–2 Average Bond

The average bond stress is used in computing anchorage or embedment of reinforcement. From any point within its length, a bar must be extended in both directions a sufficient distance to permit the development by average bond of the tensile or compressive stress in the bar at that point. Alternatively, in the case of tensile reinforcement, the force in the bar may be developed by means of a hook, or the combination of a hook plus average bond (see for example Table 13, which deals with anchorage of vertical stirrups).

The embedment length l_a is given by

$$(23)\quad l_a = \frac{\text{bar dia. (or side) x stress in bar}}{4 \times u_{av}}$$

The minimum embedment length is given when u_{av} = the permissible value U_{av}.

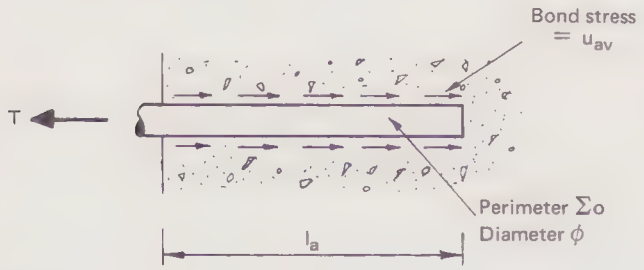


Fig. A7

From Fig. A7, $T = u_{av} \times \Sigma o \times l_a$

$$= u_{av}\, \pi\, \phi\, l_a.$$

Also for stress in bar f_s,

$$T = f_s A_s$$

$$= f_s \times \frac{\pi\phi^2}{4}.$$

Equating the two expressions for T gives Equation (23).

For bars in compression, AS CA2–1963 allows a minimum embedment length of

$$l_a = \frac{\text{bar dia. (or side) x stress in bar}}{5 \times U_{av}}.$$

This equation reflects the better bond characteristics exhibited by bars loaded in compression, as compared with tensile reinforcement.

A5 Shear Stresses and Reinforcement

A5–1 Shear Stresses

$$(24)\quad f_v = \frac{V}{bjd}.$$

f_v is the maximum shear stress acting on the cross sectional area bjd of a beam subject to a transverse shear force V. The transverse shear is in itself not usually critical insofar as the behaviour of a reinforced concrete beam is concerned. Rather, f_v is a measure of the diagonal tensile stresses which result from the combination of flexural stresses and transverse shear, and it is to limit these diagonal tensile stresses that code restrictions are applied to transverse shear.

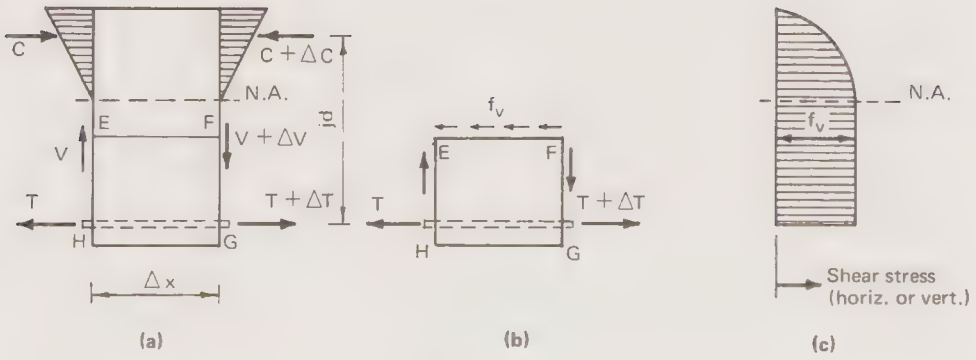


Fig. A8

Equation (24) is derived from further consideration of the elemental portion of the beam shown in Fig. A6; this portion is redrawn in Fig. A8. Consider the element EFHG, of length Δx, which lies below the neutral axis (Fig. A8 (b)). The horizontal forces acting on this element are the tensile reinforcement forces T and $T + \Delta T$, and an assumed horizontal shear stress f_v which acts over the top surface of the element.

Then for horizontal equilibrium

$$T + f_v \, \Delta x \, b = T + \Delta T,$$

from which

$$\frac{\Delta T}{\Delta x} = f_v b.$$

From the derivation of Equation (22), $\frac{\Delta T}{\Delta x} = \frac{V}{jd}$.

Therefore

$$f_v = \frac{V}{bjd}.$$

This is the value of the horizontal shear stress on all planes between the tensile reinforcement and the neutral axis, and from elementary elastic theory, the shear stress on those parts of vertical planes lying below the neutral axis has the same value. Above the neutral axis, the horizontal, and consequently the vertical shear stress reduces progressively to zero at the extreme fibre, because of the influence of horizontal compressive stresses. The shear stress distribution is shown diagrammatically in Fig. A8 (c).

When flexural tensile stresses in concrete below the neutral axis are ignored, the horizontal and vertical shears indicate the presence of maximum diagonal tensile and compressive stresses on principal planes inclined at 45° to the horizontal. Moreover from elastic theory the maximum diagonal stresses are each numerically equal to f_v, so that $f_v = \frac{V}{bjd}$ provides a direct measure of the maximum diagonal tension.

(25) $$s = \frac{A_{sv} F_{sv}}{f'_v} \times \frac{\cos\theta + \sin\theta}{b} = \frac{A_{sv} F_{sv}}{f'_v} \times \frac{B}{b}.$$ See Diagram 14.

In Equation (25), f'_v is that portion of the total vertical shear stress f_v which is to be supported by shear reinforcement, and θ is the angle of inclination between the shear reinforcement and the neutral axis.

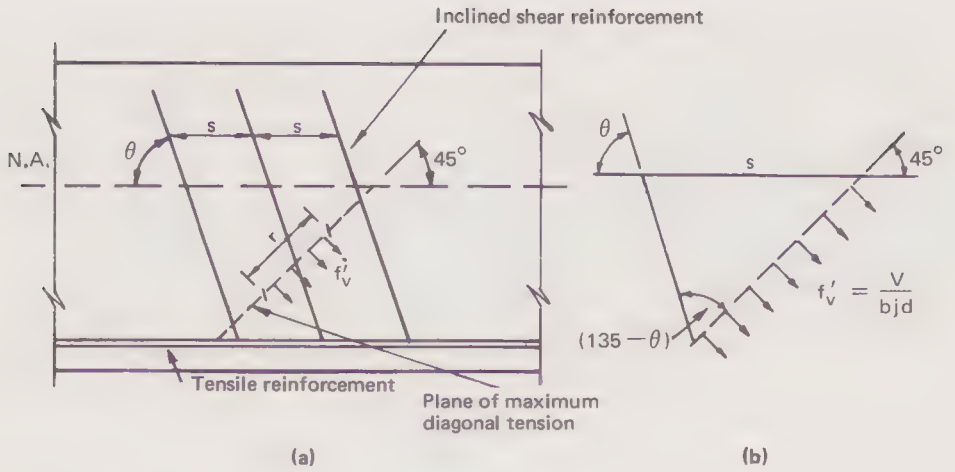


Fig. A9

Let r equal the length of that part of the plane of principal diagonal tension which is intercepted by one layer of inclined reinforcement, s the horizontal spacing between layers and θ the inclination of the reinforcement to the neutral axis.

Then $$r = \frac{s \sin \theta}{\sin (135 - \theta)}$$

$$= \frac{s \sin \theta}{\sin (45 + \theta)}$$

$$= \frac{s \sin \theta}{\sin 45 \cos \theta + \cos 45 \sin \theta}.$$

$$= \frac{\sqrt{2}\, s \sin \theta}{\cos \theta + \sin \theta}.$$

Vertical component of diagonal tensile force

$$= f'_v r b \sin 45$$

$$= \frac{f'_v s b \sin \theta}{\cos \theta + \sin \theta}.$$

Vertical component of force in reinforcement, in terms of permissible stress

$$= A_{sv}\, F_{sv} \sin \theta.$$

Equating the last two expressions gives Equation (25).

Note that the horizontal component of the diagonal tension is supported by the longitudinal reinforcement.

(26) $$\frac{V'}{jd} = \frac{A_{sv}\, F_{sv}\, (\sin \theta + \cos \theta)}{s} \text{ (lb. per in.), or}$$

$$\frac{V'}{1{,}000\ jd} = \frac{A_{sv}\, F_{sv}\, (\sin \theta + \cos \theta)}{1{,}000\ s} \text{ (kips per in.).}$$ See Table 15.

Equation (26) results from substituting $f'_v = \frac{V'}{bjd}$ in Equation (25).

B INVESTIGATION OF FLEXURAL MEMBERS

B1 Ratios of Reinforcement

For compressive reinforcement,

(27) $p' = \frac{A_{sc}}{bd}$ or $\frac{A_{sc}}{b_w d}$.

For tensile reinforcement,

(28) $p = \frac{A_{st}\, i}{bd}$ or $\frac{A_{st}\, i}{b_w d}$. (For simple bending, i = unity.)

From Equation (4),

$$A_{st} = \frac{M \text{ (in. lb.)}}{f_s j d} \quad \text{(simple bending).}$$

From Equation (5),

$$A_{st} = \frac{Ne}{adi} = \frac{1{,}000\ Ne}{f_s j d i} \quad \text{(combined bending and axial load), or}$$

$$A_{st}\, i = \frac{1{,}000\ Ne}{f_s j d}$$

Thus the general equation covering all cases of bending is

$$A_{st}\, i = \frac{\text{Moment (in. lb.)}}{f_s j d}$$

or, in terms of permissible stresses

$$A_{st}\, i = \frac{\text{Moment (in. lb.)}}{F_s j d}$$

For combined bending it has been shown that the customary design relations hold, provided A_{st} is always written together with the factor i. The same requirement therefore applies to investigation problems, for which the general equation for the ratio of tensile reinforcement is:

$$p = \frac{A_s i}{bd} \text{ or } \frac{A_s i}{b_w d}.$$

B2 Section Coefficients

B2–1 Rectangular Section

(29) $k = \sqrt{m^2 + 2q} - m$, where

(30) $m = np + (2n - 1)\, p'$, and

(31) $q = np + (2n - 1)\, p' \frac{d'}{d}$.

See Table 10.

Equation (29) also applies to T–sections in which the stress in the stem is considered. For a T–section the above expressions for m and q each contain an additional term relating to the compression flange. Consequently the derivation of the equations for this more general case is given below in Section B2–2.

(32) $$z = \frac{\frac{1}{6} + \frac{(2n-1)A_{sc}}{kbd} \times \frac{d'}{kd}\left(1 - \frac{d'}{kd}\right)}{\frac{1}{2} + \frac{(2n-1)A_{sc}}{kbd} \times \left(1 - \frac{d'}{kd}\right)},$$ See Table 11.

Where z is the ratio of the distance (zkd) between the extreme compressive fibre and the centroid of the compressive forces, to depth (kd).

The derivation of Equation (30) is given in the more general form for T–sections below (see Equation (36)).

(33) $j = 1 - zk.$ See Table 12.

This equation follows since $jd = d - zkd$.

B2–2 T–Section with Compressive Stress in Stem Included, and Compressive Reinforcement

$k = \sqrt{m^2 + 2q} - m$, where (Equation (29).)

(34) $m = np + (2n-1)\,p' + p''$, and

(35) $q = np + (2n-1)\,p'\frac{d'}{d} + p''\frac{t}{2d}$. See Table 10.

$p'' = \frac{(b - b_w)t}{b_w d}$, by definition.

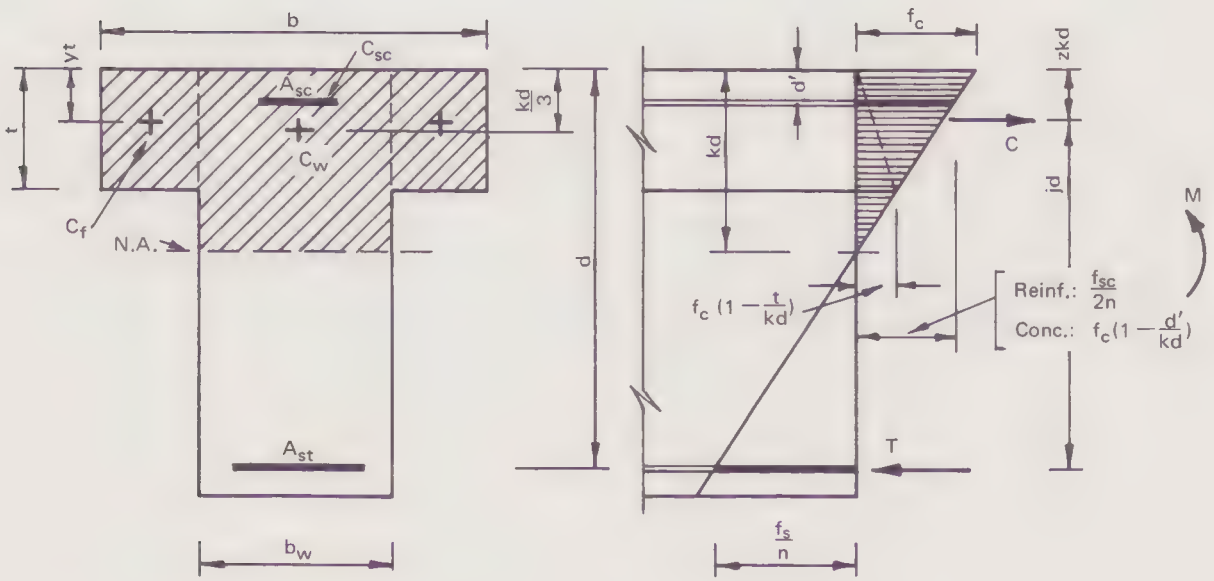


Fig. B1

Equation (29) is derived directly from the equilibrium condition C = T, where the resultant of compressive forces C is the sum of the forces in the web, reinforcement and flange respectively:

$$C = C_w + C_{sc} + C_f \qquad \text{(See Fig. B1)}$$

$$= [\tfrac{f_c}{2} b_w k d] + [f_{sc} A_{sc} - f_c(1 - \tfrac{d'}{kd})A_{sc}]$$

$$+ [\tfrac{f_c}{2}(b - b_w)t + \tfrac{f_c}{2}(1 - \tfrac{t}{kd})(b - b_w)t].$$

From similar triangles, $f_{sc} = 2n \times f_c(1 - \frac{d'}{kd})$, so that

$$C = [\tfrac{f_c}{2} b_w k d] + [(2n-1) f_c (1 - \tfrac{d'}{kd})A_{sc}]$$

$$+ [\tfrac{f_c}{2}(2 - \tfrac{t}{kd})(b - b_w)\, t].$$

Writing $C = T = f_s A_{st}$, and rearranging,

$$f_s A_{st} = \tfrac{f_c}{2}[b_w k d + (2n-1)(1 - \tfrac{d'}{kd})A_{sc} + (2 - \tfrac{t}{kd})(b - b_w)t].$$

Divide throughout by $b_w d$, and write

$$p = \frac{A_{st}}{b_w d},\ p' = \frac{A_{sc}}{b_w d} \text{ and } p'' = \frac{(b - b_w)d}{b_w d}. \text{ Then}$$

$$f_s p = \tfrac{f_c}{2}[k + (2n-1)(1 - \tfrac{d'}{kd})p' + (2 - \tfrac{t}{kd})\, p''].$$

From Equation (2), $\frac{f_s}{f_c} = \frac{n(1-k)}{k}$, so that

$$\frac{pn(1-k)}{k} = \tfrac{1}{2}[k + (2n-1)(1 - \tfrac{d'}{kd})p' + (2 - \tfrac{t}{kd})\, p''].$$

This may be rearranged in the form

$$k^2 + 2k[np + (2n-1)\, p' + p''] = 2[np + (2n-1)\, p' \frac{d'}{d} + p'' \frac{t}{2d}].$$

write $m = np + (2n-1)\, p' + p''$,

and $q = np + (2n-1)\, p' \frac{d'}{d} + p'' \frac{t}{2d}$. Then

$$k^2 + 2km = 2q.$$

Add m^2 to both sides:

$$k^2 + 2km + m^2 = 2q + m^2$$

$$(k + m)^2 = 2q + m^2$$

$$k = \sqrt{m^2 + 2q} - m.$$

(36) $$z = \frac{\tfrac{1}{6} + \frac{(2n-1)A_{sc}}{k\, b_w\, d} \times \frac{d'}{kd}(1 - \frac{d'}{kd}) + \frac{(b - b_w)t}{k\, b_w\, d} \times \frac{yt}{kd}(1 - \frac{t}{2kd})}{\tfrac{1}{2} + \frac{(2n-1)A_{sc}}{k\, b_w\, d} \times (1 - \frac{d'}{kd}) + \frac{(b - b_w)t}{k\, b_w\, d} \times (1 - \frac{t}{2kd})}.$$

The distance (zkd) is equal to the moment of the resultant of compressive forces C about the extreme compressive fibre, divided by C (see Fig. B1).

$$zkd = \frac{C_w \frac{kd}{3} + C_{sc}\, d' + C_f\, yt}{C_w + C_{sc} + C_f}$$

Substitute for C_w, C_{sc} and C_f (see derivation of Equation (29)).

$$zkd = \frac{\frac{f_c}{2} b_w kd \times \frac{kd}{3} + (2n-1) f_c \left(1 - \frac{d'}{kd}\right) A_{sc} \times d' + \frac{f_c}{2} \left(2 - \frac{t}{kd}\right)(b - b_w)t \times yt}{\frac{f_c}{2} b_w kd + (2n-1) f_c \left(1 - \frac{d'}{kd}\right) + \frac{f_c}{2} \left(2 - \frac{t}{kd}\right)(b - b_w)t}$$

$$= \frac{f_c b_w k^2 d^2 \left[\frac{1}{6} + (2n-1) \frac{A_{sc}}{k\, b_w\, d} \times \frac{d'}{kd} \left(1 - \frac{d'}{kd}\right) + \frac{(b - b_w)t}{k\, b_w\, d} \times \frac{yt}{kd} \left(1 - \frac{t}{2kd}\right)\right]}{f_c b_w kd \left[\frac{1}{2} + (2n-1) \frac{A_{sc}}{k\, b_w\, d} \left(1 - \frac{d'}{kd}\right) + \frac{(b - b_w)t}{k\, b_w\, d} \times \left(1 - \frac{t}{2kd}\right)\right]}.$$

Divide both sides of the equation by kd and cancel $f_c b k d$, which gives Equation (36).

By setting the flange area $(b - b_w)t = 0$ in Equation (36), Equation (32) for z for rectangular sections is obtained directly.

(37) $$j = 1 - \left[\frac{\frac{k^2}{6} + \frac{(2n-1)A_{sc}}{b_w d} \times \frac{d'}{d} \left(1 - \frac{d'}{kd}\right) + \frac{(b - b_w)t}{b_w d} \times \frac{yt}{d} \left(1 - \frac{t}{2kd}\right)}{\frac{k}{2} + \frac{(2n-1)A_{sc}}{b_w d} \left(1 - \frac{d'}{kd}\right) + \frac{(b - b_w)t}{b_w d} \left(1 - \frac{t}{2kd}\right)}\right].$$

Equation (37) follows directly from Equations (33) and (36).

B2–3 T–Section with Compressive Stress in Stem Included, and No Compressive Reinforcement

When compressive reinforcement is not present in a T–section with stress in stem included, the terms involving A_{sc} or p' are eliminated from Equations (34), (35), (36) and (37) for m, q, z and j respectively.

(38) $$z = \frac{\frac{1}{6} + \frac{(b - b_w)t}{k\, b_w\, d} \times \frac{yt}{kd} \left(1 - \frac{t}{2kd}\right)}{\frac{1}{2} + \frac{(b - b_w)t}{k\, b_w\, d} \times \left(1 - \frac{t}{2kd}\right)}.$$

(39) $$j = 1 - \left[\frac{\frac{k^2}{6} + \frac{(b - b_w)t}{b_w d} \times \frac{yt}{d} \left(1 - \frac{t}{2kd}\right)}{\frac{k}{2} + \frac{(b - b_w)t}{b_w d} \times \left(1 - \frac{t}{2kd}\right)}\right].$$

When a section is investigated, m and q are evaluated and k determined as usual from Table 10 (or Equation (29)). Then j may be calculated directly from Equation (39), or alternatively, Tables 11 and 12 may be used to evaluate first z and then j. The latter approach, which is demonstrated in Examples (15) and (20), is based on the close similarity between Equations (32) and (38). To enter Table 11, $(b - b_w)t$ (the flange area) is substituted for A_{sc}, and yt for d'. Exact similarity is evidently only possible if $\frac{yt}{d} = \frac{t}{2d}$ in Equation (38), that is if $y = \frac{1}{2}$, whereas in practical sections y is always somewhat less than $\frac{1}{2}$ (see Table 8). The error in the value of j which results from this inequality is negligible, however.

B2–4 T–Section with Compressive Stress in Stem Neglected

(40) $$k = \frac{\frac{n A_{st}\, i}{bt} + \frac{1}{2} \times \frac{t}{d}}{\frac{n A_{st}\, i}{bt} + 1}.$$ (For simple bending, i = unity.)

Equation (40) is derived from the requirement that the neutral axis must pass through the centroid of the combined compressive concrete and tensile reinforcement areas. The tensile reinforcement area is the transformed area, $nA_{st}i$, and the concrete area, bt. Taking moments about the neutral axis,

$$bt\,(kd - \tfrac{t}{2}) = n\,A_{st}\,i\,(d - kd).$$

$$\text{Then } k - \tfrac{1}{2} \times \frac{t}{d} = \frac{n\,A_{st}\,i}{bt}\,(1 - k),$$

and Equation (40) follows.

B3 Stress Determination

B3–1 Tensile Stresses

(41) $$f_s = \frac{M\text{ (in. lb.)}}{jd\,A_{st}} = \frac{1{,}000\,M\text{ (in. kips)}}{j\,d\,A_{st}}\text{ (simple bending).}$$

(42) $$f_s = \frac{1{,}000\,N}{j\,A_{st}\,i} \times \frac{e}{d}\text{ (combined bending and axial load).}$$

Equations (41) and (42) follow directly from Equations (4) and (5).

B3–2 Concrete Compressive Stresses

(43) $$f_c = \frac{f_s}{n} \times \frac{k}{1-k}.$$

Equation (43) follows directly from Equation (2).

B3–3 Compressive Reinforcement Stresses

For a modular ratio of 2n applied to the compressive reinforcement,

(44) $$f_{sc} = 2f_s \times \frac{k - \frac{d'}{d}}{1-k}.$$

Equation (44) follows from the derivation of Equations (9) and (10).

As shown in Examples 13, 16, 18, 21 and 23, f_{sc} when computed from Equation (44) frequently exceeds the permissible value F_{sc}. The computation may then be repeated using a modular ratio for the compressive reinforcement which is reduced from 2n to $2n'$, in order to reduce the computed value of f_{sc}. For modular ratio $2n'$,

$$m = p + (2n' - 1)\,p' + p'',$$

$$q = p + (2n' - 1)\,p'\frac{d'}{d} + p''\frac{t}{2d}, \text{ and}$$

(45) $f_{sc} = \frac{2n'}{n} f_s \times \frac{k - \frac{d'}{d}}{1 - k}$.

Equation (45) may be derived from similar triangles in Fig. A3, using $\frac{f_{sc}}{2n'}$, and $\frac{f_s}{n}$ in place of $\frac{f_{sc}}{2n}$ and $\frac{F_s}{n}$.

Then $\frac{f_{sc}}{2n'}(d - kd) = \frac{f_s}{n}(kd - d')$,

$$\text{or } f_{sc} = \frac{2n'}{n} f_s \times \frac{kd - d'}{d - kd}$$

$$= \frac{2n'}{n} f_s \times \frac{k - \frac{d'}{d}}{1 - k}.$$

C1 Concentrically Loaded Compression Members

For short tied compression members ($\frac{h}{D} \leqslant 15$):

(46) $$P = \frac{0.30\ F'_c\ A_g + F_{sc}\ A_s}{1{,}000}$$ See Tables 16, 17.

For short helically reinforced compression members ($\frac{h}{D} \leqslant 15$):

(47) $$P = \frac{0.30\ F'_c\ A_k + F_{sc}\ A_s + 27{,}000\ A_b}{1{,}000}$$ See Tables 17, 19.

For long compression members, $\frac{h}{D} > 15$, and the axial loads given by Equations (46) and (47) must be reduced by the multiplication factor $(1.6 - 0.04\ \frac{h}{D})$.

Equations (46) and (47) do not derive from elastic theory, but are semi-empirical relationships based on ultimate load behaviour.

C2 Eccentrically Loaded Compression Members

In the derivation of design and investigation equations for eccentrically loaded compression members, the following assumptions are made:

(i) the tensile strength of concrete is ignored, and
(ii) the modular ratio n is applicable to both compressive and tensile reinforcement.

C2–1 Entire Area of Section in Compression — $k \geqslant 1$

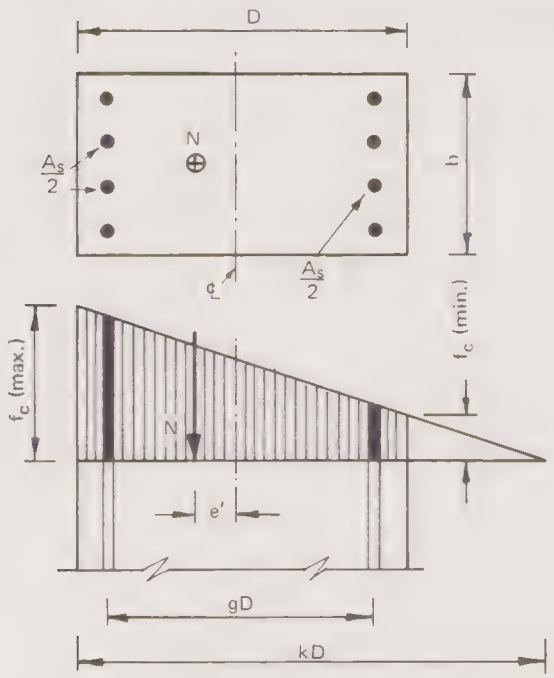


Fig. C1

I = moment of inertia about axis of bending.

r = radius of gyration about axis of bending.

k = ratio of distance kD to section depth D.

(48) f_c (max.) $= \frac{1{,}000\ N}{A_g} \times \frac{1}{1+(n-1)p}\ (1+\frac{D^2}{2r^2} \times \frac{e'}{D})$.

(49) f_c (min.) $= \frac{1{,}000\ N}{A_g} \times \frac{1}{1+(n-1)p}\ (1-\frac{D^2}{2r^2} \times \frac{e'}{D})$.

Equations (48) and (49) give the maximum and minimum stresses at the extreme concrete fibres, each being the sum of:

(i) the uniform stress due to concentric load N, and
(ii) the flexural stress due to bending moment Ne'.

The uniform stress in the concrete under axial loading is given by

$$\frac{1{,}000\ N}{A_g} \times \frac{A_g}{A_g+(n-1)\ A_{sc}} = \frac{1{,}000\ N}{A_g} \times \frac{1}{1+(n-1)p}, \text{ and}$$

the maximum flexural stress by

$$\pm \frac{1{,}000\ Ne'}{\frac{2\,I}{D}}$$

where $I = r^2\ A_g\ [1+(n-1)p]$ for the transformed section.

Addition of the above two expressions for stresses gives Equations (48) and (49), which are valid for any symmetrical section.

Expressions for the parameters $\frac{D^2}{2r^2}$, and $\frac{1{,}000\ N}{f_c\,bD}$ or $\frac{1{,}000\ N}{f_c\,D^2}$, are given below for each of the three cross-sectional shapes treated in Diagrams 21 – 23.

(i) Rectangular section with reinforcement symmetrically arranged at opposite faces (Fig. C1).

For transformed section $I = \frac{b\,D^2}{12} + (n-1)\ pbd\ (\frac{gD}{2})^2$.

Therefore $\frac{D^2}{2r^2} = \frac{D^2}{2} \times \frac{A}{I}$ (where A is the transformed area)

$$= \frac{D^2}{2} \times \frac{bD+(n-1)\ pbD}{\frac{bD^2}{12}+(n-1)pbD\ \frac{(gD)^2}{4}}$$

$$= \frac{1+(n-1)\ p}{\frac{1}{6}+\frac{1}{2}\ (n-1)\ pg^2}.$$

(50) $$\frac{1{,}000\ N}{f_c\ b\ D} = \frac{1}{\frac{1}{1+np}+\frac{2\frac{e'}{D}}{\frac{1}{3}g^2np}}$$

From Equation (48), $\frac{1{,}000\ N}{f_c\,bD} = \frac{1+(n-1)p}{1+\frac{D^2}{2r^2} \times \frac{e'}{D}}$.

Substitute for $\frac{D^2}{2r^2}$: $\frac{1{,}000\ N}{f_c\ b\ D} = \frac{1+(n-1)p}{1+\frac{e'}{D}\ [\frac{1+(n-1)p}{\frac{1}{6}+\frac{1}{2}\ (n-1)pg^2}]}$

$$= \frac{1}{\frac{1}{1+(n-1)p}+\frac{e'}{D}\ [\frac{1}{\frac{1}{6}+\frac{1}{2}\ (n-1)pg^2}]}$$

which is identical with Equation (50).

Equation (50) takes into account the area of concrete which is displaced by compressive reinforcement. When concrete displacement is ignored, Equation (50) becomes:

(51) $$\frac{1{,}000\,N}{f_c\,bD} = \frac{1}{\dfrac{1}{1+np} + \dfrac{2\frac{e'}{D}}{\frac{1}{3}+npg^2}}$$ See Diagram 21.

The following relationships for helically reinforced sections are equivalent to those given above for rectangular sections and may be derived in a similar manner:

(ii) Square section with reinforcement equidistantly spaced within the helix:

$$\frac{D^2}{2r^2} = \frac{1+(n-1)p}{\frac{1}{6}+\frac{1}{4}(n-1)pg^2}\text{ , and}$$

(52) $$\frac{1{,}000\,N}{f_c\,D^2} = \frac{1}{\dfrac{1}{1+np} + \dfrac{2\frac{e'}{D}}{\frac{1}{3}+\frac{1}{2}g^2np}}$$, corresponding to Equation (51). See Diagram 22.

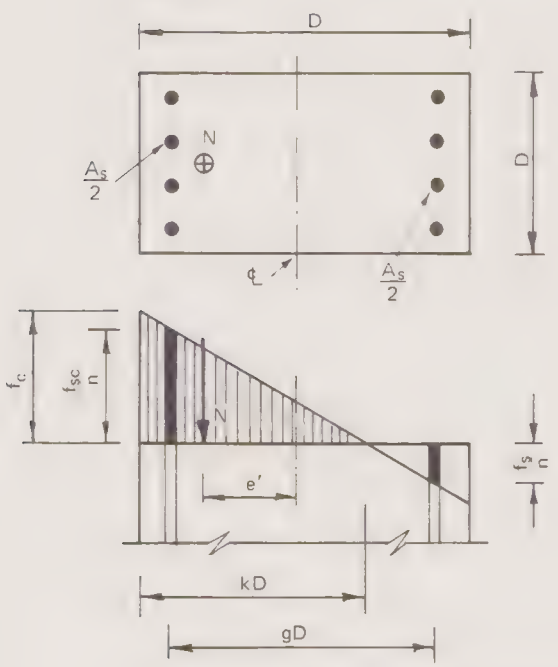
(iii) Round section with reinforcement equidistantly spaced within the helix.

$$\frac{D^2}{2r^2} = \frac{1+(n-1)p}{\frac{1}{8}+\frac{1}{4}(n-1)pg^2}\text{ , and}$$

(53) $$\frac{1{,}000\,N}{f_c\,D^2} = \frac{\frac{\pi}{4}}{\dfrac{1}{1+np} + \dfrac{2\frac{e'}{D}}{\frac{1}{3}+\frac{1}{2}g^2np}}$$, corresponding to Equation (51). See Diagram 23.

C2–2 Tension over Part Area of Section – Cracked Section with k < 1

Fig. C2



In the following equations the transformed area of compressive reinforcement has been taken as nA_s, rather than the strictly correct value, $(n-1)A_s$. This has been done to avoid difficulties in presentation of Diagrams 21 – 23.

(i) Rectangular section with reinforcement symmetrically arranged at opposite faces (Fig. C2).

(54) $\frac{1{,}000\,N}{f_c\,bD} = \frac{k^2 + np\,(2k-1)}{2k}$. See Diagram 21.

For equilibrium between external and internal forces acting on the cross section,

$$1{,}000\,N = \frac{f_c}{2}\,bkd - f_s\,\frac{A_s}{2} + f_{sc}\,\frac{A_s}{2}\ .$$

Divide throughout by $f_c\,bD$:

$$\frac{1{,}000\,N}{f_c\,bD} = \frac{k}{2} + \frac{f_{sc} - f_s}{f_c} \times \frac{p}{2}\ .$$

From similar triangles in Fig. C2,

$$\frac{\frac{f_s}{n}}{gD + \frac{D - gD}{2} - kD} = \frac{f_c}{kD}\ ,$$

$$\text{so that } f_s = nf_c\,(\frac{1+g}{2k} - 1).$$

$$\text{Similarly } f_{sc} = nf_c\,(1 - \frac{1-g}{2k}).$$

$$\text{Therefore } \frac{f_{sc} - f_s}{f_c} = n\,(2 - \frac{1}{k}),$$

$$\text{and } \frac{1{,}000\,N}{f_c\,bD} = \frac{k}{2} + n(2 - \frac{1}{k})\frac{p}{2}$$

$$= \frac{k^2 + np\,(2k-1)}{2k}\ .$$

(55) $\frac{1{,}000\,Ne'}{f_c\,bD^2} = \frac{k^2\,(1 - \frac{2}{3}k) + g^2\,np}{4k}$. See Diagram 21.

Take moments about the centroidal axis of the section:

$$1{,}000\,Ne' = \frac{f_c}{2}\,bkd\,(\frac{D}{2} - \frac{kD}{3}) + f_{sc}\,\frac{A_s}{2}\,\frac{gD}{2} + f_s\,\frac{A_s}{2}\,\frac{gD}{2}\ .$$

Divide throughout by $f_c\,bD^2$:

$$\frac{1{,}000\,Ne'}{f_c\,bD^2} = \frac{k}{2}\,(\tfrac{1}{2} - \frac{k}{3}) + (\frac{f_{sc} + f_s}{f_c})\,\frac{gp}{4}\ .$$

From the derivation of Equation (54), $\frac{f_{sc} + f_s}{f_c} = \frac{ng}{k}$.

$$\text{Therefore } \frac{1{,}000\,Ne'}{f_c\,bD^2} = \frac{k}{2}\,(\tfrac{1}{2} - \frac{k}{3}) + \frac{g^2\,np}{4k}$$

$$= \frac{k^2\,(1 - \frac{2}{3}k) + g^2\,np}{4k}$$

For helically reinforced sections, the relationships corresponding to Equations (54) and (55) are:

(ii) Square section with reinforcement equidistantly spaced within the helix:

(56) $\frac{1{,}000\,N}{f_c\,D^2} = \frac{k^2 + np\,(2k-1)}{2k}$.

(57) $\frac{1{,}000\,Ne'}{f_c\,D^3} = \frac{k^2\,(1 - \frac{2}{3}k) + \frac{1}{2}\,g^2\,np}{4k}$.

See Diagram 22.

(iii) Round section with reinforcement equidistantly spaced within the helix:

(58) $$\frac{1{,}000\ N}{f_c D^2} = \frac{\pi}{8k}\left[\frac{4k^2 - 4k + 3}{1.5\pi}\sqrt{k - k^2} + \frac{2k - 1}{\pi}\ \text{arc cos}\ (1 - 2k) + np\ (2k - 1)\right].$$

(59) $$\frac{1{,}000\ Ne'}{f_c D^3} = \frac{\pi}{16k}\left[\frac{-16k^3 + 24k^2 - 2k - 3}{6\pi}\sqrt{k - k^2} + \frac{1}{4\pi}\ \text{arc cos}\ (1 - 2k) + \tfrac{1}{2} g^2\ np\right].$$

See Diagram 23.

C2–3 Stresses

(60) $f_s = nf_c\left(\frac{1+g}{2k} - 1\right)$ (maximum tensile stress).

(61) $f_{sc} = nf_c\left(1 - \frac{1-g}{2k}\right)$ (maximum compressive stress).

Equations (60) and (61) are applicable to all symmetrical sections, for all values of k. They follow directly from the derivation of Equation (54).

For small k, the maximum reinforcement tensile stress may approach the permissible value. The limiting value of k at which $f_{sc} = F_{sc}$, is given by

(62) $$k = \frac{1+g}{2\left(\frac{F_s}{nF_c} + 1\right)}.$$ See Table 20.

Equation (62) is obtained by rearranging Equation (60), expressed in terms of permissible stresses.

D1 Reinforced Square Spread Footings

D1–1 Basis for Design

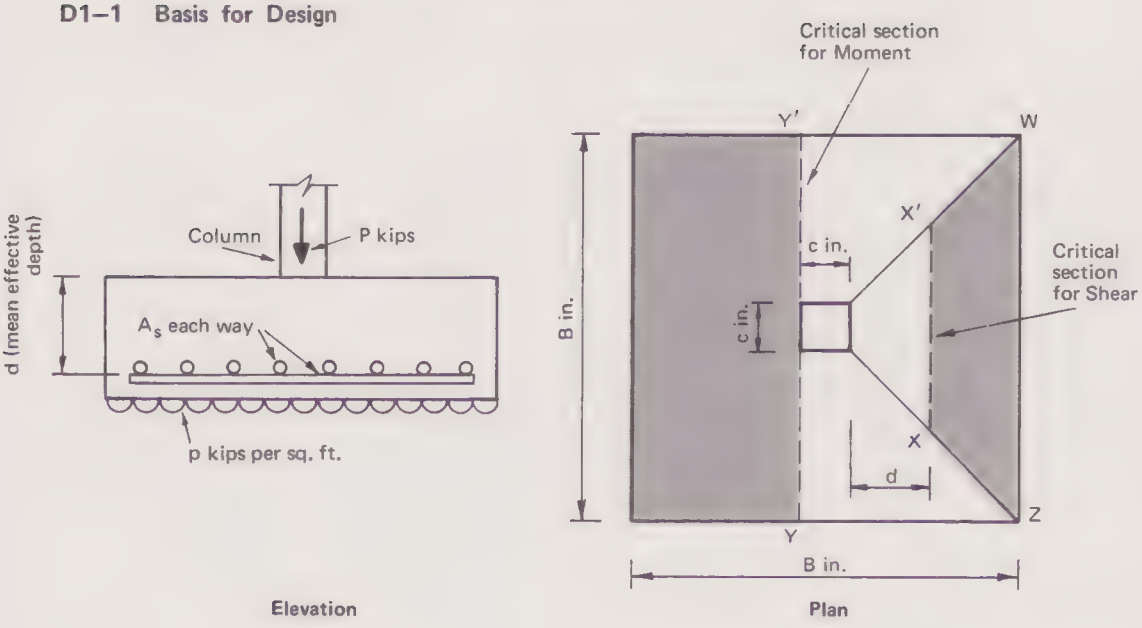


Fig. D1

In accordance with AS CA2–1963, the critical sections for shear and moment in reinforced concrete footings on soil are those shown in Fig. D1.

In evaluating the nett transverse shear force acting on the footing, the designer may choose either to include or to ignore the footing self weight, the choice being a matter of soil mechanics rather than of reinforced concrete design. Denoting the nominal bearing pressure under the footing as p (kips per sq. ft.), the self weight of the footing as w (kips per sq. ft. of plan area), and the column load as P (kips), then:

(i) For footing weight included:

$$\text{Total actual load on soil} = \text{column load} + \text{footing weight} = P + \frac{wB^2}{144} = \frac{pB^2}{144} \text{ kips.}$$

$$\text{Nett transverse loading on footing} = (p - w) \text{ kips per sq. ft.}$$

(ii) For footing weight ignored:

$$\text{Total nominal load on soil} = \text{column load } P = \frac{pB^2}{144} \text{ kips.}$$

$$\text{Total actual load on soil} = \text{column load} + \text{footing weight} = P + \frac{wB^2}{144} = \frac{(p+w)B^2}{144} \text{ kips.}$$

$$\text{Nett transverse loading on footing} = p \text{ kips per sq. ft.}$$

The data in Table 24 have been computed on the basis that the footing weight is ignored. Table values are therefore accurate (within the minor limitations expressed below) for designs with footing weight ignored, and conservative for designs in which footing weight is included.

The column dimension c given in Table 24 is the side dimension of a square column which will support the tabulated load $\frac{pB^2}{144}$ in accordance with Equation (46). The assumed properties of the column are:

Reinforcement $A_s = 0.04\ Ag$, with $F_{sc} = 23{,}000$ psi.

Concrete F'_c = the lesser of 5,000 psi. or $1.67 \times F'_c$ for footing.

The latter restriction on column concrete strength limits the bearing stress under a concentrically loaded column to $0.5 \times F'_c$ for footing.
Values of c in Table 24 are rounded to the nearest whole number.

D1–2 Effective Depth

The effective depth based on shear at section XX′ is given by

$$(63)\quad d = \frac{-c(R+3) + \sqrt{c^2(R+3)^2 + R(R+6)(B^2 - c^2)}}{2(R+6)},$$

where $R = \frac{1{,}000\,p}{48\,F_v j}$.

From Equation (24), $d = \frac{V}{F_v j b}$

$$= \frac{V}{F_v j (c + 2d)}.$$

V is the shear force on the tributary area XX′WZ (see Fig. D1), for transverse loading p when the footing self weight is ignored.

$$V = 1{,}000\,p \times \tfrac{1}{4}\left[\left(\frac{B}{12}\right)^2 - \left(\frac{c+2d}{12}\right)^2\right].$$

Therefore $$d = \frac{250\,p\left[\left(\frac{B}{12}\right)^2 - \left(\frac{c+2d}{12}\right)^2\right]}{F_v j\,(c+2d)}$$

$$= \frac{R}{12} \times \frac{[B^2 - (c+2d)^2]}{c+2d}.$$

This equation may be rearranged as the quadratic

$$4d^2(R+6) + 4dc(R+3) - R(B^2 - c^2) = 0,$$

of which the positive root is given by Equation (63).

The effective depth based on bending moment at section YY′ is given by

$$(64)\quad d = \sqrt{QB}\,(B - c),$$

where $Q = \frac{425p}{576K}$.

From Equation (1), $d = \sqrt{\frac{M_R}{Kb}}$, in which M_R = the externally applied moment M for balanced conditions.

$$M = 0.85^{*} \times 1{,}000\ p \times \frac{B(B-c)}{144 \times 2} \times \frac{(B-c)}{4} \text{ in. lb.}$$

$$\text{Then } d = \sqrt{\frac{850\ p}{1{,}152\ K}\ B(B-c)^2}$$

$$= \sqrt{QB}\ (B-c).$$

For the range of footings given in Table 24, the larger value of d is always given by Equation (63). The smallest practical value of d has been taken as 6 inches.

D1–3 Tensile Reinforcement

$$A_s = \frac{M}{F_s j d} \qquad \text{(Equation (4)).}$$

$$(65)\quad A_s = \frac{\phi V}{4\ U j d} \qquad \text{(local bond at section YY').}$$

$$(66)\quad A_s = \frac{\phi M}{4\ l_a\ U_{av}\ j d} \qquad \text{(average bond at section YY', assuming bars are not hooked at ends).}$$

In these Equations:

$$M = \frac{425}{576}\ p\ B(B-c)^2;$$

$$V = 0.85 \times 1{,}000\ p \times \frac{B(B-c)}{144 \times 2}$$

$$= \frac{425}{144}\ p\ B(B-c);$$

ϕ = diameter of bars;

l_a = embedment length shown in Fig. D1.

Equation (65) derives from Equation (22), considered together with the following relationships for any number of bars n:

$$A_s = n\frac{\pi\phi^2}{4}, \text{ and}$$

$$\Sigma o = n\pi\phi.$$

Equation (66) is obtained by eliminating F_s between Equations (4) and (23).

To compute tabulated values of d from Equation (63) and A_s from Equations (4), (65) or (66), values of j for balanced conditions have been used. This procedure is slightly conservative since all footings listed in Table 24 are under-reinforced, and in consequence the true values of j are greater than the balanced values. For A_s values the degree of conservatism has been reduced by using the average effective depth (Fig. D1) in the solution of Equation (4), rather than the more strictly correct depth to the upper layer of reinforcement.

* For two-way reinforced footings, 0.85 of the actual moment may be used in computations (see AS CA2–1963).

D2 Unreinforced Rectangular Mass Footings

The design of unreinforced footings differs from that of reinforced sections in that account is taken of the ability of the concrete to resist small tensile stresses. The self weight may be either ignored, or included in design computations.

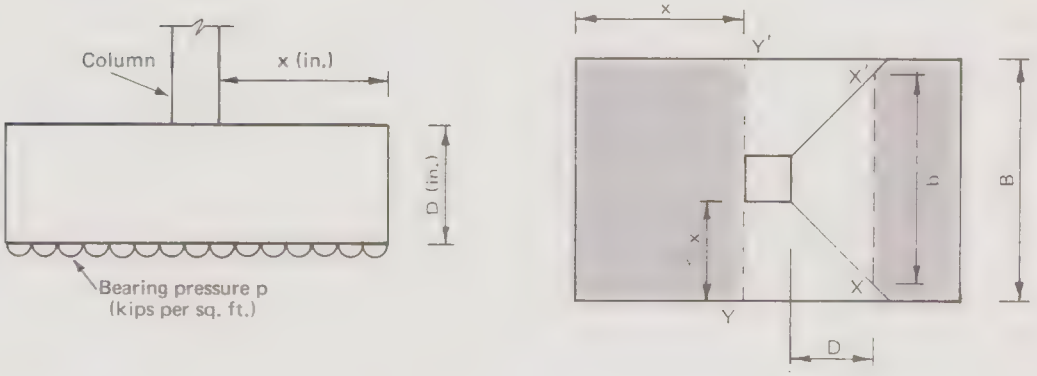


Fig. D2

D2–1 Self Weight of Footing Included

The critical sections for moment and shear in plain rectangular concrete footings on soil or rock are those shown in Fig. D2.

The total footing depth based on bending moment at Section YY′ is given by

$$(67)\quad D = \tfrac{1}{2}\left[\sqrt{\left(\frac{0.261x^2}{F_{ct}}\right)^2 + \frac{83.3\,x^2 p}{F_{ct}}} - \frac{0.261\,x^2}{F_{ct}}\right]. \qquad \text{See Table 25.}$$

Soil pressure = p kips per sq. ft.

= 6.94 p psi.

Footing weight = 150 lb. per cubic ft.

= 0.0868 D lb. per sq. in. of plan area.

Moment at Section YY′, per inch width

$$= (6.94\,p - 0.0868D) \times \frac{x^2}{2},$$

and modulus of section per inch width

$$= \frac{D^2}{6}.$$

Therefore $F_{ct} = (6.94\,p - 0.0868\,D) \times \frac{3x^2}{D^2}$.

This expression may be rearranged as the quadratic

$$D^2 + \frac{.261\,x^2}{F_{ct}} - \frac{20.83\,p\,x^2}{F_{ct}} = 0,$$

of which the positive root is given by Equation (67).

The total depth based on shear at Section XX′ is given by

(68) $D = \frac{V}{bF_v'}$, where V is the nett shear force on the tributary area lying outside Section XX′.

Equation (68) is the rearranged expression for permissible average shear stress, $F_v = \frac{V}{bD}$.

For the range of footings included in Table 25, the critical (greater) value of D is always given by Equation (67).

D2–2 Self Weight of Footing Ignored

For D based on bending:

(69) $D = \sqrt{\frac{20.83\,p}{F_{ct}}}\,x = \beta x.$ See Table 25.

Equation (69) follows if the footing weight is omitted in the derivation of Equation (67).

Design and Investigation of Flexural Members

Simple Bending and Bending Combined with Axial Load

The design procedure presented here for flexural members is essentially that of determining a concrete section which has suitable dimensions and sufficient reinforcement to resist a known loading, without exceeding permissible flexural stresses in concrete or steel. Examples 1 to 5 demonstrate the design procedure for simple bending, and Examples 6 to 9 introduce the combination of simple bending and axial load.

In Examples 1 and 2, dealing with a slab and a rectangular beam, respectively, the design problem has been set up in some detail as a guide for the student.

Determination of the required depth of slabs having balanced reinforcement and subject to simple bending involves only the use of Table 2, from which the effective depth can be selected when the permissible working stresses and the value of n are known. The dimensions of rectangular beams can be determined, when similar information is given, from Tables 1 and 4. K is taken from Table 1 and used in computing $F = \frac{M}{K}$. With F known, b and d may be selected from Table 4.

Tensile reinforcement is determined from $A_{st} = \frac{M}{ad}$, or for balanced conditions only, from $A_{st} = pbd$, where a and p are given in Table 1. Use of the tabulated values of $a = \frac{F_s j}{1{,}000}$ also leads to accurate values of A_{st} for balanced conditions, but is slightly conservative for under-reinforced sections.

For slabs, a suitable size and spacing of reinforcement is determined from Table 3(a). Bar perimeters for bond calculation, and details of secondary reinforcement for slabs are given in Tables 3(b) and 3(c), respectively. For the rectangular beam, the number and size of bars is determined from Table 5(b) and the sum of perimeters Σo for bond calculation from Table 5(c). Minimum web width b to maintain adequate cover to, and spacing of, the selected bars, is checked in Table 6.

Whereas in Examples 1 and 2, the slab or beam dimensions are determined as a matter of design, in Examples 3 to 9 it is assumed that the section dimensions have already been established, and it remains in each instance to determine the reinforcement only.

The problem of determining the compressive and tensile reinforcement in doubly reinforced rectangular beams, as in Example 3, is solved by computing $A_{sc} = \frac{M - KF}{cd}$ and $A_{st} = \frac{M}{ad}$. The numerator in the expression for A_{sc} represents that part of the external moment which the concrete section is unable to carry when stressed to the permissible limit, and which must therefore be taken by compressive reinforcement. Values of c are taken from Table 7. Again, as for rectangular beams reinforced in tension only, A_{st} based upon a will be approximate only, but sufficiently accurate for most purposes. For deep beams with large amounts of compressive reinforcement it is more accurate to compute A_{st} as the sum of two areas by the alternative method demonstrated in Example 3, viz:

$$A_{st} = pbd + \frac{1{,}000\,(M - KF)}{F_s\,(d - d')}$$

If the stress in the stem is neglected, the design of a T–section subject to simple bending is essentially the same as the design of a rectangular beam except that the value of K is taken from Table 8 instead of Table 1. Tensile reinforcement is computed from $A_{st} = \frac{1{,}000\,M}{F_s\,j\,d}$, where j is also obtained from Table 8.

When the stress in the stem of a T–section is included and compressive reinforcement is or is not required, the design procedure is similar to that of a rectangular beam if the web and flange are considered separately (see Example 5). When considering the web, values of K, a and F are taken from Tables 1 and 4, and when considering the flange the values of K and j are taken from Table 8.

There is no fundamental difference between the design of flexural members for simple bending and for those cases of combined bending and axial load in which tension exists over a considerable part of the cross section. Bending moment M, used in the design equations for simple bending, is replaced by Ne in which N is the axial load on the section and e the distance from the resultant of the tensile stresses to the equivalent eccentric load N that may be substituted for M and N. For example, the equations $A_{st} = \frac{M}{ad}$ or $A_{st} = \frac{1{,}000\,M}{F_s\,jd}$ for simple bending, become $A_{st} = \frac{Ne}{adi}$ or $A_{st} = \frac{1{,}000\,Ne}{F_s\,jdi}$

respectively for combined bending and axial load, where $i = \frac{1}{1 - \frac{id}{e}}$. When these substitutions are made, all equations customarily used for simple bending are applicable to combined bending and axial load. Values of a are taken from Table 1 and of j from Table 8, depending on whether the member is a rectangular or T—section, and values of i for axial compression are taken from Table 9. Examples 6 to 8 illustrate the design procedure for sections subject to bending combined with axial compression, while Example 9 deals with axial tension. For the latter case values of i are not taken from Table 9, but must be computed.

When the eccentricity e is relatively small, the procedure for the design of eccentrically loaded columns should be used, as set out in Examples 26 and 27.

Shear Reinforcement

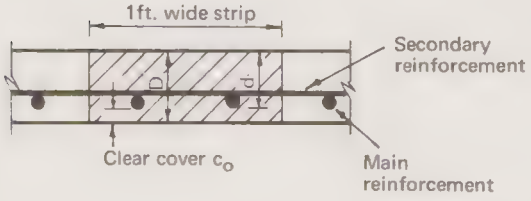
Shear reinforcement is required in flexural members whenever the total transverse shear stress $f_v = \frac{V}{bjd}$ exceeds the permissible shear capacity of the concrete alone. Two types of shear reinforcement are commonly used, namely, stirrups or hoops, and inclined bars, and accordingly design data have been provided for these two cases. Example 10 deals with vertical stirrups and Example 11 with a combination of the two types.

It is first necessary to determine the distribution of f_v along the beam length, so that the permissible concrete stresses and consequently the excess shear f'_v can be evaluated. Where stirrups are to be used the quantity $\frac{A_{sv}\, F_{sv}}{1{,}000} \times \frac{B}{b}$ is then determined; the value of $A_{sv}\, F_{sv}$ being computed in the case of inclined stirrups, or taken from Table 13 for vertical stirrups. With f'_v and $\frac{A_{sv}\, F_{sv}}{1{,}000} \times \frac{B}{b}$ known, spacing s is read from Diagram 14. For inclined bars, s is obtained from Table 15 when the quantity $\frac{V'}{1{,}000\ jd}$ is known.

Example 1
Design of Slab, Simple Bending

A simply supported one way slab carries a uniform superimposed live loading w_L of 125 lb. per sq. ft. If the effective span L is 15 ft, determine:

(a) a suitable slab thickness D;
(b) area of main tensile reinforcement A_{st};
(c) a suitable bar size and spacing for main reinforcement;
(d) a suitable bar size and spacing for secondary reinforcement.



Given :

F'_c = **3,000 psi.**
F_s = **20,000 psi.**
L = **15 ft.**
w_L = **125 lb. per sq. ft.**

(1) Assume a slab thickness D = 6 in. (corresponding to maximum $\frac{L}{D} = 30$ from AS CA2–1963).

Then self weight of slab (dead loading) w_D = 75 lb. per sq. ft.
Live loading w_L = 125 lb. per sq. ft.
Total $w_D + w_L = w$ = 200 lb. per sq. ft.

(2) Compute maximum bending moment

$$M = \frac{wL^2}{8}$$
$$= \frac{200 \times 15^2 \times 12}{8 \times 1{,}000}$$
$$= 67.5 \text{ in. kips.}$$

(3) From Table 2, for F_s/F'_c = 20,000/3,000 and M = 67.5: interpolation gives required d = 4.7 in.*

Calculate available d assuming a bar size of #6 (or ϕ 6) and concrete cover $c_o = \frac{3}{4}$ in. (c_o from AS CA2–1963, for maximum aggregate size of $\frac{3}{4}$ in.).

$$\text{Available d} = 6 - \tfrac{1}{2} \times \tfrac{3}{4} - \tfrac{3}{4}$$
$$= 4.9 \text{ in.}$$

Therefore depth of slab as assumed in (1) is satisfactory.

(4) From Table 1, for F_s/F'_c = 20,000/3,000 : $a = \frac{F_s j}{1{,}000} = 17.02$ and j = 0.851.

Therefore, area of main reinforcement required, $A_{st} = \frac{M}{ad}$

$$= \frac{67.5}{17.02 \times 4.9}$$
$$= 0.81 \text{ sq. in.}$$

(5) From Tables 3(a) and 3(b), for 1 ft. width of slab, #6 bars at $6\frac{1}{2}$ in. centres gives

A_{st} = 0.816 sq. in.
and Σo = 4.35 in.

(Note from Table 3(c) that #6 bars at $6\frac{1}{2}$ in. centres complies with maximum spacing and minimum area requirements for main reinforcement in slabs.)

---

* Evaluation of d, or of concrete dimensions generally, to a greater accuracy than 0.1 in. is seldom warranted in the design of reinforced concrete.

(6) Checking local bond stress :

$$\text{support shear } V = \frac{200 \times 15}{2}$$
$$= 1{,}500 \text{ lb.}^*$$

For $\Sigma o = 4.35$ in. compute local bond stress $u = \frac{V}{\Sigma o j d}$

$$= \frac{1{,}500}{4.35 \times 0.851 \times 4.9}$$
$$= 82.7 \text{ psi.}$$

This stress is less than the permissible value U = 350 psi. (from AS CA2–1963).

(7) Shear is not generally critical for slabs, and is not checked here. Computations for shear are given in Examples 2, 10 and 11.

(8) Assuming that the slab is exposed to the sun, then from AS CA2–1963, the minimum area of deformed secondary reinforcement permitted is 0.20%, and the maximum spacing between bars, 5d. From Table 3(c), for D = 6 in. and p = 0.20%, select #4 bars at 16 in. centres (from Part (ii) of table), or #5 bars at 24 in. centres (from Part (i) of table).**

Moreover the area of secondary reinforcement must be not less than 20% of the area of main reinforcement, that is 0.20 x 0.816 = 0.163 sq. in. per ft. width of slab. Corresponding bar spacings are #4 bars at 14 in. centres of #5 at 22 in. centres, which are closer than those evaluated above.

Therefore choose #4 bars at 14 in. centres or #5 bars at 22 in. centres for secondary reinforcement, to be placed on top of the main reinforcement.

---

* The effective span for moment (L) and the critical span for shear between the faces of the supports, are here assumed equal.

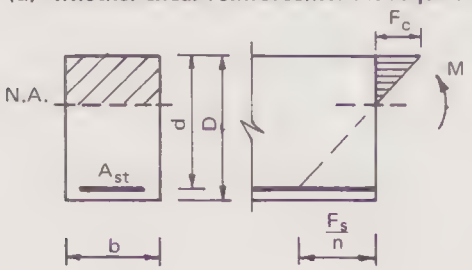
** # 3 bars are not available currently in Australia.

Example 2
Rectangular Beam, Simple Bending

A simply supported rectangular concrete beam carries a uniform superimposed live loading w_L of 700 lb. per ft. run. The effective span, L, between supports is 30 ft. and the beam width b is required to be 12 in.

Assuming lateral restraint at supports only, determine :

(a) the minimum effective depth d required to avoid the necessity for compressive reinforcement;
(b) area of main tensile reinforcement A_{st};
(c) a suitable number and size of bars for tensile reinforcement;
(d) whether shear reinforcement is required.



Given:

F'_c = 3,000 psi.
F_s = 20,000 psi.
L = 30 ft.
b = 12 in.
w_L = 700 lb. per ft. run.

(1) From AS CA2–1963, check need for stress reduction owing to conditions of lateral restraint:*

$$\frac{L}{b} = \frac{30 \times 12}{12}$$

$$= 30$$

Therefore no reduction is necessary in the value of F_c, so that from Table 1, for $F_s/F'_c = 20{,}000/3{,}000$: K = 257, j = 0.851, p = 0.0151 and a = 17.02.

(2) Assume an effective depth d = 18 in. and an overall depth D = 20 in., say.

$$\text{Compute beam self weight } w_D = \frac{150 \times 20 \times 12}{144}$$

$$= 250 \text{ lb. per ft. run.}$$

$$\text{Total loading } w = 250 + 700$$

$$= 950 \text{ lb. per ft. run.}$$

(3) Compute the maximum bending moment

$$M = \frac{wL^2}{8}$$

$$= \frac{950 \times 30^2 \times 12}{1{,}000 \times 8}$$

$$= 1{,}280 \text{ in. kips.}$$

(4) Compute d from $M = \frac{Kbd^2}{1{,}000}$:

$$d = \sqrt{\frac{1{,}000 \times 1{,}280}{257 \times 12}}$$

$$= 20.4 \text{ in.}$$

(5) Alternatively compute F from M = KF :

$$F = \frac{1{,}280}{257}$$

$$= 4.98$$

From Table 4, for F = 4.98 and b = 12 in. : d = 20.4 in.

(6) The value of d = 20.4 in. is greater than that assumed in calculating w_D in (2) above. A more accurate estimate is necessary.

Assuming D = 23 in.,

$$w_D = \frac{150 \times 23 \times 12}{144}$$

$$= 288 \text{ lb. per ft. run,}$$

$$\text{and total loading } w = 288 + 700$$

$$= 988 \text{ lb. per ft. run.}$$

(7) Repeat step (3) :

$$\text{maximum } M = \frac{988 \times 30^2 \times 12}{1{,}000 \times 8}$$

$$= 1{,}330 \text{ in. kips.}$$

(8) Repeat step (5) (or (4)) to determine d :

$$\text{compute } F = \frac{1{,}330}{257}$$

$$= 5.18$$

From Table 4, for F = 5.18 and b = 12 in.:

for a balanced section d = 20.8 in., say 21 in.

(9) Compute

$$A_{st} = \frac{M}{ad}$$

$$= \frac{1{,}330}{17.02 \times 21}$$

$$= 3.72 \text{ sq. in.}$$

---

* For $\frac{L}{b} > 30$, AS CA2–1963 requires a reduction in permissible concrete compressive stresses.

Example 2 (ctd.)

(10) Alternatively compute $A_{st} = pbd^*$

$= 0.0151 \times 12 \times 21$

$= 3.81$ sq. in.

(11) From Table 5(b): 4 # 9 bars gives $A_{st} = 3.98$ sq. in.

From Table 6, the minimum web width required for 4 # 9 bars together with ϕ3 stirrups, say, is 11.5 in., which $< b = 12$ in.
Also, allowing 1 in. cover to reinforcement, the minimum value of D is

$D = d + 1\frac{1}{8} \times \frac{1}{2} + \frac{3}{8} + 1$

$= 23$ in., which

confirms the assumed value of D used to determine w_D in step (6).

(12) Checking local bond stress :

compute support shear $V = \frac{988 \times 30}{1{,}000 \times 2}$

$= 14{,}800$ lb.

From Table 5(c), for 4 # 9 bars : $\Sigma o = 14.1$ sq. in.

Local bond stress $u = \frac{V}{\Sigma o jd}$

$= \frac{14{,}800}{14.1 \times 0.851 \times 21}$

$= 58.7$ psi.

Note that if 2 #9 bars are curtailed short of the support, then $\Sigma o = 7.07$ sq. in. and $u = 117$ psi. Therefore, for either 4 #9 or 2 #9 bars, u is less than the maximum permissible value $U = 350$ psi. given in AS CA2–1963.

(13) Checking shear stress at support : $f_v = \frac{V}{bjd}$

$= \frac{14{,}800}{12 \times 0.851 \times 21}$

$= 69.0$ psi.

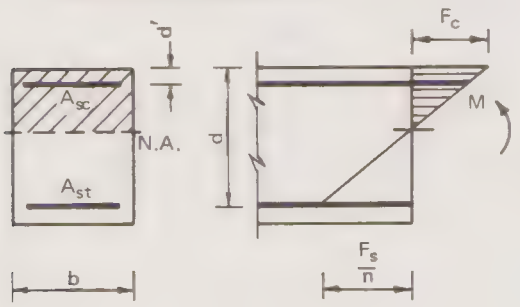
No shear reinforcement is required for a simple beam when $f_v \leqslant 0.02 F'_c + 20$ (= 80 psi. here) $\leqslant 90$ psi.

* This calculation using p from Table 1 gives A_{st} correctly for balanced conditions only. The coefficient a may be used as in step (9) to derive A_{st} when the section is not balanced. This procedure is not exact but is sufficiently accurate for practical design purposes.

Example 3
Rectangular Section, Simple Bending — Compressive Reinforcement

A rectangular section is limited in size for architectural reasons to a maximum effective depth d = 15 in. and breadth b = 9 in. If the applied maximum moment is M = 600 in. kips determine :

(a) whether the concrete section is adequate without compressive reinforcement;
(b) compressive reinforcement, A_{sc}, if required;
(c) tensile reinforcement, A_{st}.



Given:

F_s = 20,000 psi.
F_{sc} = 16,500 psi.
F'_c = 3,000 psi.
d = 15 in. d' = 2 in.
b = 9 in.
M = 600 in. kips.

(1) From Table 1, for F_s/F'_c = 20,000/3,000 : K = 257.
From Table 4, for b x d = 9 x 15 : F = 2.03.
Compute moment of resistance KF = 257 x 2.03
= 522 in. kips.
Applied moment M = 600 in. kips.
Then M — KF = 78 in. kips.

Compressive reinforcement is required when (M — KF) is positive since this is the residual moment not taken by the concrete.

(2) From Table 7, for $F_s/F_{sc}/F'_c$ = 20,000/16,500/3,000 and $\frac{d'}{d}$ = 0.133 :
c = 13.5.

$$\text{Compute } A_{sc} = \frac{M - KF}{cd}$$
$$= \frac{78}{13.5 \times 15}$$
$$= 0.39 \text{ sq. in.}$$

(3) From Table 1, for F_s/F'_c = 20,000/3,000 : a = 17.02 and p = 0.0151.

$$\text{Compute } A_{st} = \frac{M^*}{ad}$$
$$= \frac{600}{17.02 \times 15}$$
$$= 2.35 \text{ sq. in.}$$

(4) Alternatively tensile reinforcement to balance concrete = pbd
= 0.0151 x 9 x 15
= 2.04 sq. in.,

and tensile reinforcement to balance compressive steel $= \frac{1{,}000\,(M - KF)}{F_s\,(d - d')}$
$= \frac{1{,}000 \times 78}{20{,}000 \times 13}$
= 0.30 sq. in.

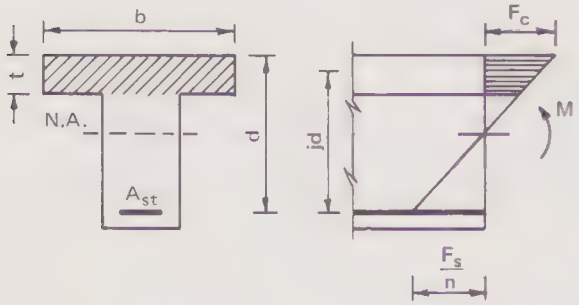
Then A_{st} = 2.04 + 0.30
= 2.34 sq. in.

---

* As indicated in the introduction to the design of flexural members, p.45, computation of A_{st} based upon a is approximate only, but sufficiently accurate for most design purposes.

Example 4
T-Section, Simple Bending — Stress in Stem Neglected

For a T-section of specified dimensions and subject to a bending moment M, determine:
(a) whether the concrete section is adequate without compressive reinforcement;
(b) tensile reinforcement, A_{st}. Neglect compressive stress below flange.



Given:

F_s = 20,000 psi.
F'_c = 3,000 psi.
d = 25 in.
t = 5 in.
b = 40 in.
M = 3,000 in. kips.

(1) From Table 8, for F_s/F'_c = 20,000/3,000 and $\frac{t}{d}$ = 0.20 :

K = 191.

From Table 4, for b x d = 40 x 25 : F = 25.0 (twice value for b = 20 in., say).

Compute moment of resistance KF = 191 x 25.0
= 4,775 in. kips.
Applied moment M = 3,000 in. kips.
Then M — KF = −1,775 in. kips.

No compressive reinforcement is required since (M — KF) is negative.

(2) From Table 8, for F_s/F'_c = 20,000/3,000 and $\frac{t}{d}$ = 0.20 :

j = 0.91.

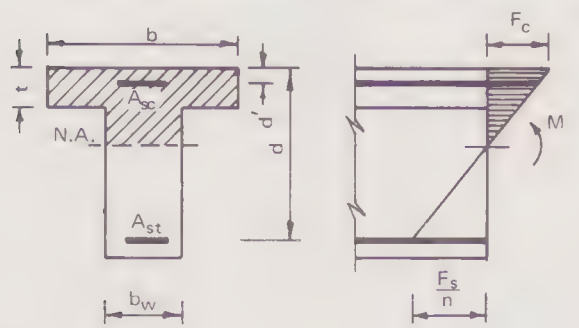
Compute $A_{st} = \frac{1{,}000M}{F_s jd}$

$= \frac{1{,}000 \times 3{,}000}{20{,}000 \times 0.91 \times 25}$

= 6.60 sq. in.

Example 5
T-Section, Simple Bending — Stress in Stem Included, Compressive Reinforcement

For a T-section of specified dimensions and subject to a bending moment M, determine:
(a) whether the concrete section is adequate without compressive reinforcement;
(b) compressive reinforcement A_{sc}, if required;
(c) tensile reinforcement A_{st}. Include compressive stress below flange.



Given:

F_s = 20,000 psi.
F_{sc} = 16,500 psi.
F'_c = 3,000 psi.
d = 25 in. t = 5 in.
d' = 2 in.
b = 40 in. b_w = 15 in.
M = 6,000 in. kips.

(1) Web — From Table 1, for $F_s/F'_c = 20{,}000/3{,}000 : K_w = 257.$

From Table 4, for $b_w \times d = 15 \times 25 : F_w = 9.38.$

Flange — From Table 8, for $F_s/F'_c = 20{,}000/3{,}000$ and $\frac{t}{d} = 0.20 : K_f = 191.$

From Table 4, for $(b - b_w) \times d = 25 \times 25 : F_f = 15.6$

(add values for b = 12 in. and b = 13 in., say).

$$M = 6{,}000 \text{ in. kips.}$$
$$\text{Compute } K_w F_w = 257 \times 9.38$$
$$= 2{,}410 \text{ in. kips, and}$$
$$K_f F_f = 191 \times 15.6$$
$$= \underline{2{,}980} \text{ in. kips.}$$
$$\text{Then } M - \Sigma(KF) = 610 \text{ in. kips.}$$

Compressive reinforcement is required since $(M - \Sigma(KF))$ is positive.

(2) From Table 7, for $F_s/F_{sc}/F'_c = 20{,}000/16{,}500/3{,}000$ and $\frac{d'}{d} = 0.080 : c = 14.2.$

$$\text{Then } A_{sc} = \frac{M - \Sigma(KF)}{cd}$$
$$= \frac{610}{14.2 \times 25}$$
$$= 1.72 \text{ sq. in.}$$

(3) **Web.** From Table 1, for $F_s/F'_c = 20{,}000/3{,}000 : a_w = 17.02$

$$\text{Then } A_{sw} = \frac{K_w F_w}{a_w d}$$
$$= \frac{2{,}410}{17.02 \times 25}$$
$$= 5.66 \text{ sq. in.}$$

Flange. From Table 8, for $F_s/F'_c = 20{,}000/3{,}000$ and $\frac{t}{d} = 0.20 : j_f = 0.91.$

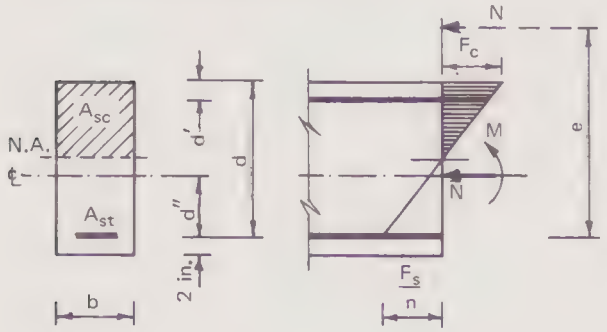
$$\text{Then } A_{sf} = \frac{1{,}000 (M - K_w F_w)}{F_s j_f d}$$
$$= \frac{1{,}000 \times 3{,}590}{20{,}000 \times 0.91 \times 25}$$
$$= 7.90 \text{ sq. in.,}$$
$$\text{and } A_{st} = A_{sw} + A_{sf}$$
$$= 5.66 + 7.90$$
$$= 13.56 \text{ sq. in.}^*$$

* For $A_{st} = 13.56$ sq. in., 14 #9 bars would be required, say, arranged in 3 horizontal layers to maintain proper spacing and cover (see Table 6). A_{st} could be reduced by choosing reinforcement with $F_s = 25{,}000$ psi. or 30,000 psi. for this beam.

Example 6
Rectangular Section, Combined Bending and Axial Load — Compressive Reinforcement

For a rectangular section of specified dimensions and subject to a bending moment M combined with an axial load N, determine :

(a) whether the concrete section is adequate without compressive reinforcement;
(b) compressive reinforcement, A_{sc}, if required;
(c) tensile reinforcement, A_{st}.



Given:
F_s = 20,000 psi.
F'_c = 3,000 psi.
d = 22 in. d' = 2 in.
d'' = 10 in.
b = 12 in.
M = 1,290 in. kips.
N = 30 kips.

(1) Compute eccentricity e measured from tensile reinforcement :

$$e = \frac{M}{N} + d''$$
$$= \frac{1,290}{30} + 10$$
$$= 53 \text{ in.}$$
$$\frac{e}{d} = \frac{53}{22}$$
$$= 2.41.$$

(2) From Table 1, for F_s/F'_c = 20,000/3,000 : K = 257 and j = 0.851.
From Table 4, for b x d = 12 x 22 : F = 5.81.

Compute Ne = 30 x 53
= 1,590 in. kips, and
KF = 257 x 5.81
= 1,493 in. kips.
Then Ne — KF = 97 in. kips.

Compressive reinforcement is required since (Ne — KF) is positive.

(3) From Table 7, for $F_s/F_{sc}/F'_c$ = 20,000/16,500/3,000 and $\frac{d'}{d}$ = 0.091 : c = 14.0.

$$\text{Then } A_{sc} = \frac{Ne - KF}{cd}$$
$$= \frac{97}{14.0 \times 22}$$
$$= 0.32 \text{ sq. in.}$$

(4) From Table 9, for $\frac{e}{d}$ = 2.41 and j = 0.851 : i = 1.55.

From Table 1, for F_s/F'_c = 20,000/3,000 : a = 17.02.

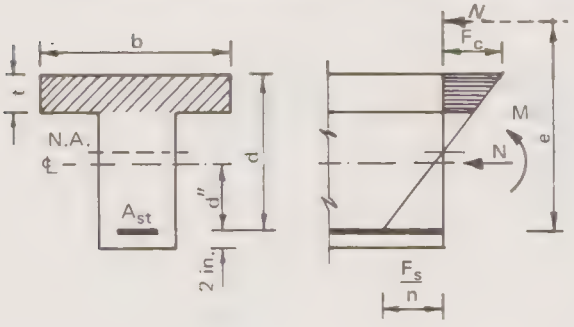
$$\text{Then } A_{st} = \frac{Ne}{adi}$$
$$= \frac{1,590}{17.02 \times 22 \times 1.55}$$
$$= 2.74 \text{ sq. in.}$$

Example 7
T–Section, Combined Bending and Axial Load – Stress in Stem Neglected

For a T–section of specified dimensions subject to a bending moment M combined with an axial load N, determine :

(a) whether the concrete section is adequate without compressive reinforcement;

(b) tensile reinforcement, A_{st}. Neglect compressive stress below flange.



Given:

F_s = 20,000 psi.
F'_c = 3,000 psi.
d = 24 in. t = 5 in.
d'' = 11 in.
b = 36 in.
M = 2,800 in. kips.
N = 70 kips.

(1) Compute eccentricity e measured from tensile reinforcement:

$$e = \frac{M}{N} + d''$$
$$= \frac{2,800}{70} + 11$$
$$= 51 \text{ in.}$$
$$\frac{e}{d} = \frac{51}{24}$$
$$= 2.13 \text{ in.}$$

(2) From Table 8, for F_s/F'_c = 20,000/3,000 and $\frac{t}{d}$ = 0.208 : K = 195.

From Table 4, for b x d = 36 x 24 : F = 20.7.

Compute Ne = 70 x 51
= 3,570 in. kips, and
KF = 195 x 20.7
= 4,040 in. kips.
Then Ne – KF = –470 in. kips.

No compressive reinforcement is required since (Ne – KF) is negative.

(3) From Table 8, for F_s/F'_c = 20,000/3,000 and $\frac{t}{d}$ = 0.208 : j = 0.905.

From Table 9, for $\frac{e}{d}$ = 2.13 and j = 0.905 : i = 1.74.

$$\text{Then } A_{st} = \frac{1,000 \text{ Ne}}{F_s jdi}$$
$$= \frac{1,000 \times 3,570}{20,000 \times 0.905 \times 24 \times 1.74}$$
$$= 4.72 \text{ sq. in.}$$

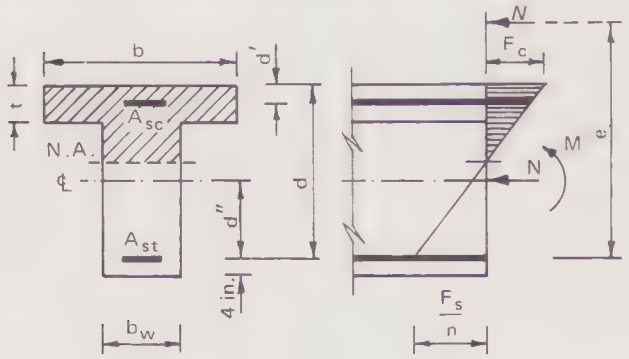
Example 8
T–Section, Combined Bending and Axial Load – Stress in Stem Included, Compressive Reinforcement

For a T–section of specified dimensions subject to a bending moment M combined with an axial load N, determine :

(a) whether the concrete section is adequate without compressive reinforcement;

(b) compressive reinforcement A_{sc}, if required;

(c) tensile reinforcement A_{st}. Include compressive stress below flange.



Given:

F_s = 20,000 psi.
F_{sc} = 16,500 psi.
F'_c = 3,000 psi.
d = 24 in. t = 5 in.
d' = 2 in. d'' = 11 in.
b = 36 in. b_w = 15 in.
M = 3,300 in. kips.
N = 150 kips.

(1) Compute eccentricity e measured from tensile reinforcement:

$$e = \frac{M}{N} + d''$$
$$= \frac{3{,}300}{150} + 11$$
$$= 33 \text{ in.}$$
$$\frac{e}{d} = \frac{33}{24}$$
$$= 1.38.$$

(2)

Web —
- From Table 1, for F_s/F'_c = 20,000/3,000 : K_w = 257.
- From Table 4, for $b_w \times d$ = 15 x 24 : F_w = 8.64.

Flange —
- From Table 8, for F_s/F'_c = 20,000/3,000 and $\frac{t}{d}$ = 0.208 : K_f = 195.
- From Table 4, for $(b - b_w) \times d$ = 21 x 24 : F_f = 12.1.

Compute Ne = 150 x 33
= 4,950 in. kips,
$K_w F_w$ = 257 x 8.64
= 2,220 in. kips, and
$K_f F_f$ = 195 x 12.1
= 2,360 in. kips.

Then $Ne - \Sigma(KF)$ = 370 in. kips.

Compressive reinforcement is required since $(Ne - \Sigma(KF))$ is positive.

(3) From Table 7, for $F_s/F_{sc}/F'_c$ = 20,000/16,500/3,000 and $\frac{d'}{d}$ = 0.083 : c = 14.2.

$$\text{Then } A_{sc} = \frac{Ne - \Sigma(KF)}{cd}$$
$$= \frac{370}{14.2 \times 24}$$
$$= 1.09 \text{ sq. in.}$$

(4) **Web.** From Table 1, for $F_s/F'_c = 20{,}000/3{,}000$: $j_w = 0.851$ and $a_w = 17.02$.

From Table 9, for $\frac{e}{d} = 1.38$ and $j_w = 0.851$: $i_w = 2.60$.

$$\text{Then } A_{sw} = \frac{K_w F_w}{a_w d i_w}$$

$$= \frac{2{,}220}{17.02 \times 24 \times 2.60}$$

$$= 2.09 \text{ sq. in.}$$

Flange. From Table 8, for $F_s/F'_c = 20{,}000/3{,}000$ and $\frac{t}{d} = 0.208$: $j_f = 0.905$.

From Table 9, for $\frac{e}{d} = 1.38$ and $j_f = 0.905$: $i_f = 2.91$.

$$\text{Then } A_{sf} = \frac{1{,}000(Ne - K_w F_w)}{F_s j_f d i_f}$$

$$= \frac{1{,}000 \times 2{,}730}{20{,}000 \times 0.905 \times 24 \times 2.91}$$

$$= 2.16 \text{ sq. in.,}$$

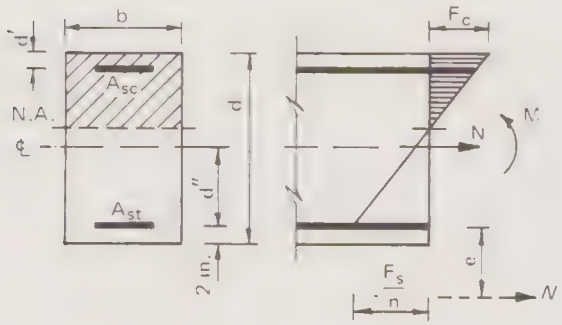
$$\text{and } A_{st} = A_{sw} + A_{sf}$$

$$= 2.09 + 2.16$$

$$= 4.25 \text{ sq. in.}$$

Example 9
Rectangular Section, Combined Bending and Axial Load (Tension) — Compressive Reinforcement

For a rectangular section of specified dimensions subject to a bending moment M combined with an axial tension N, determine :
(a) whether the concrete section is adequate without compressive reinforcement;
(b) compressive reinforcement A_{sc}, if required;
(c) tensile reinforcement, A_{st}.



Given:
F_s = 20,000 psi.
F_{sc} = 16,500 psi.
F'_c = 3,000 psi.
d = 22 in. d' = 2 in.
d'' = 10 in.
b = 14 in.
M = 2,740 in. kips.
N = −45 kips (negative because axial tension).

(1) Compute eccentricity e measured from tensile reinforcement:

$$e = \frac{M}{N} + d''$$

$$= \frac{2{,}740}{-45} + 10$$

$$= -50.9 \text{ in.}$$

Example 9 (ctd.)

(2) From Table 1, for F_s/F'_c = 20,000/3,000 : K = 257.
From Table 4, for b x d = 14 x 22 : F = 6.78.

Compute Ne = (−45) x (−50.9)
= 2,290 in. kips, and
KF = 257 x 6.78
= 1,740 in. kips.

Then Ne − KF = 550 in. kips.

Compressive reinforcement is required since (Ne − KF) is positive.

(3) From Table 7, for $F_s/F_{sc}/F'_c$ = 20,000/16,500/3,000 and $\frac{d'}{d}$ = 0.091 : c = 14.0.

$$\text{Then } A_{sc} = \frac{Ne - KF}{cd}$$
$$= \frac{550}{14.0 \times 22}$$
$$= 1.79 \text{ sq. in.}$$

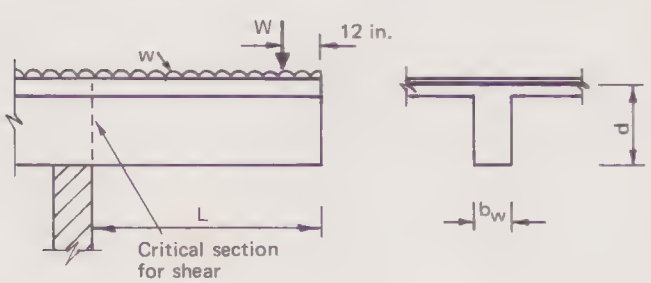
(4) From Table 1, for F_s/F'_c = 20,000/3,000 : j = 0.851 and a = 17.02.
Since e is negative, i cannot be taken from Table 9 and must be computed:

$$i = \frac{1}{1 - \frac{id}{e}}$$
$$= \frac{1}{1 - \frac{0.851 \times 22}{-50.9}}$$
$$= 0.73.$$

$$\text{Then } A_{st} = \frac{Ne}{adi}$$
$$= \frac{2,290}{17.02 \times 22 \times 0.73}$$
$$= 8.4 \text{ sq. in.}$$

Example 10
Shear Reinforcement — Vertical Stirrups

A continuous T-beam is cantilevered a distance L beyond the external face of its end support. If the cantilever supports a uniformly distributed load w (including self weight) and a concentrated load W near its tip, determine a suitable arrangement of shear reinforcement using vertical stirrups.



Given:
F'_c = 3,000 psi.
F_s = 20,000 psi.*
F_{sv} = 20,000 psi.
b_w = 12 in.
d = 24 in.
W = 24,000 lb.
w = 1,500 lb. per ft. run
L = 6 ft.

* The permissible tensile stress F_s in the main reinforcement is given as a guide for the selection of j (Table 1) and of A_{sv} F_{sv} for stirrups (Table 13) only.

(1) Compute maximum shear at the face of the support, the critical section for shear

$$V = 24{,}000 + 1{,}500 \times 6$$
$$= 33{,}000 \text{ lb.}$$

From Table 1, for $F_s/F'_c = 20{,}000/3{,}000$: $j = 0.851$.

$$\text{Compute } f_v = \frac{V}{b_w jd}$$
$$= \frac{33{,}000}{12 \times 0.851 \times 24}$$
$$= 135 \text{ psi.}$$

Also, immediately to the left of W, $V = 24{,}000 + 1{,}500 \times 1$

$$= 25{,}500 \text{ lb.,}$$
$$\text{and } f_v = \frac{25{,}500}{12 \times 0.851 \times 24}$$
$$= 104 \text{ psi.}$$

(2) From AS CA2–1963, for $f_v > 0.02\ F'_c + 20 = 80$ psi. (or a maximum of 90 psi.), but $f_v \leqslant 0.04\ F'_c + 40 = 160$ psi. (or a maximum of 180 psi.), shear reinforcement must be provided to resist shear stresses in excess of $0.02\ F'_c + 20 = 80$ psi. (or a maximum of 90 psi.).

The excess shear stresses are : at support, $f'_v = 135 - 80$

$$= 55 \text{ psi.,}$$
and at W, $f'_v = 104 - 80$
$$= 24 \text{ psi.}$$

(3) The choice will be made between $\phi 2$, $\phi 3$ and $\phi 4$ stirrups.

For $\phi 2$ stirrups:
From Table 13, for $F_s/F'_c = 20{,}000/3{,}000$ and $d = 24$ in.:

$$\frac{A_{sv} F_{sv}}{1{,}000} = 1.96 \text{ kips.}$$
$$\text{Compute } \frac{A_{sv} F_{sv}}{1{,}000} \times \frac{B}{b_w} = 1.96 \times \frac{1.00}{12}$$
$$= 0.164 \text{ kips per in.}$$

From Diagram 14, for $f'_v = 55$ psi. : $s = 3.0$ in. at support,
and for $f'_v = 24$ psi. : $s = 6.8$ in. at W.

For $\phi 3$ stirrups: $\frac{A_{sv} F_{sv}}{1{,}000} = 4.42$ kips,

$$\frac{A_{sv} F_{sv}}{1{,}000} \times \frac{B}{b_w} = 0.368 \text{ kips per in.}$$

From Diagram 14 : $s = 6.8$ in. at support,
and $s = 15$ in. at W.

For $\phi 4$ stirrups: $\frac{A_{sv} F_{sv}}{1{,}000} = 7.23$ kips, *

$$\frac{A_{sv} F_{sv}}{1{,}000} \times \frac{B}{b_w} = 0.603 \text{ kips per in.}$$

From Diagram 14: $s = 11$ in. at support,
and $s = 25$ in. at W.

Between the point load and the end of the cantilever, the total shear stress is, by inspection, negligible, and theoretically no stirrups are required.

* Note from Table 13 that $\frac{A_{sv} F_{sv}}{1{,}000} = 7.23$ kips is less than the maximum value of 7.85 kips for $\phi 4$ stirrups, indicating that for $d = 24$ in., $\phi 4$ stirrups are being used inefficiently. Fully efficient anchorage would be obtained with #4 stirrups.

(4) The maximum spacing permitted for vertical stirrups when required as shear reinforcement, is $\frac{d}{2} = 12$ in. (AS CA2–1963, and Diagram 14). $\phi 4$ stirrups would therefore be used uneconomically when restricted to 12 in. spacing, and on this basis are not selected for this particular example.

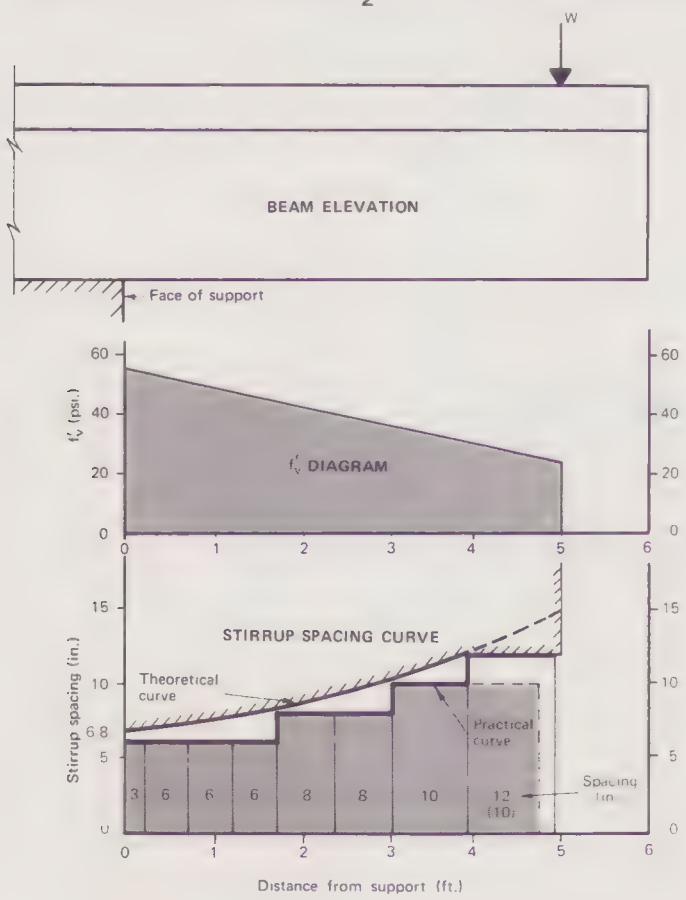
$\phi 2$ and $\phi 3$ stirrups, however, can be used at full capacity, except immediately adjacent to the point load where the theoretical spacing for $\phi 3$ stirrups is 15 in. The smaller number of $\phi 3$ stirrups is the more satisfactory choice here, being more robust than $\phi 2$ stirrups and allowing better access for placing and compacting concrete.

(5) A number of methods may be used to establish a practical arrangement of stirrups based upon the theoretical spacings calculated above. A graphical method is used here to provide a visual demonstration of the procedure.

In the diagram below, the beam length is plotted horizontally and the theoretical stirrup spacings vertically, the Code restriction on spacing at a maximum of 12 in. being imposed adjacent to W.

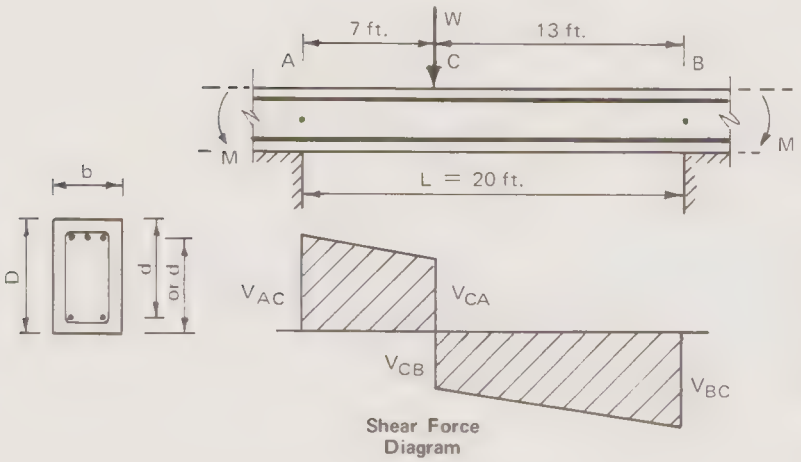
Stirrups are represented on the diagram by vertical lines of height equal to the spacing chosen. The aim then becomes simply to assign spacings so that the stepped curve joining the tops of the vertical stirrup lines approximates to the theoretical spacing curve. Note that it is conservative to maintain the practical stepped curve below the theoretical curve, as has been done in this example. In order to obtain a safe practical arrangement of stirrups it is only necessary to ensure that the area under each horizontal step does not exceed that under the corresponding section of the theoretical curve.

Here, the first practical spacing has been chosen as 6 in. The first stirrup is then located at $\frac{6}{2} = 3$ in. from the face of the support, and therefore in the centre of the area which it serves. The last spacing between the seventh and eighth stirrups is shown as 12 in., but could equally well have been maintained at 10 in. without requiring the use of any additional stirrups. A ninth stirrup would be required only if the space between the eighth stirrup and W exceeded $\frac{12}{2} = 6$ in. No stirrups are provided to the right of W.



Example 11
Shear Reinforcement — Vertical Stirrups and Inclined Bars

A rectangular beam of clear span L between supports and which forms part of a continuous structure, is subject to equal end moments M and an asymmetrically located point load W. Determine a suitable arrangement of shear reinforcement for the beam.



Shear Force Diagram

Given:

$b = 20$ in.
$d = 40$ in. $D = 44$ in.
$L = 20$ ft.
$F'_c = 3{,}000$ psi.
$F_s = 20{,}000$ psi.
$F_{sv} = 20{,}000$ psi.
$W = 180{,}000$ lb.

(1) Compute beam self weight :

$$w_D = \frac{150 \times 44 \times 20}{144}$$

$$= 920 \text{ lb. per ft. run.}$$

For equal applied end moments M, the shear force V_{AC} at the face of the left hand support A is

$$V_{AC} = \frac{13W}{L} + \frac{L}{2} \times w_D$$

$$= \frac{13 \times 180{,}000}{20} + \frac{20}{2} \times 920$$

$$= 126{,}200 \text{ lb.}$$

At the face of the right hand support B,

$$V_{BC} = \frac{7 \times 180{,}000}{20} + \frac{20}{2} \times 920$$

$$= 72{,}200 \text{ lb.}$$

Immediately to the left of C,

$$V_{CA} = 126{,}200 - 7 \times 920$$

$$= 119{,}800 \text{ lb.,}$$

and to the right of C,

$$V_{CB} = 72{,}200 - 13 \times 920$$

$$= 60{,}200 \text{ lb.}$$

(2) From Table 1, for $F_s/F'_c = 20{,}000/3{,}000$: $j = 0.851$.

Compute gross shear stress $f_v = \frac{V}{bjd}$ corresponding to shear forces evaluated above:

for $V_{AC} = 126{,}200$ lb.,

$$f_v = \frac{126{,}200}{20 \times 0.851 \times 40}$$

$$= 185 \text{ psi. ;}$$

$V_{BC} = 72{,}200$ lb.,

$$f_v = \frac{72{,}200}{20 \times 0.851 \times 40}$$

$$= 106 \text{ psi. ;}$$

$V_{CA} = 119{,}800$ lb.,

$$f_v = \frac{119{,}800}{20 \times 0.851 \times 40}$$

$$= 176 \text{ psi. ;}$$

and $V_{CB} = 60{,}200$ lb.,

$$f_v = \frac{60{,}200}{20 \times 0.851 \times 40}$$

$$= 88.4 \text{ psi.}$$

Over the length of beam AC, f_v varies linearly between 185 psi. and 176 psi., both values being greater than $f_v = 0.04\ F'_c + 40 = 160$ psi. but less than $f_v = 0.08\ F'_c + 80 = 320$ psi. Therefore at least two types of shear reinforcement are necessary (see AS CA2—1963), and the total shear stress f_v must be resisted by this reinforcement. Select two types, namely vertical stirrups and inclined bars. Each type must then be capable of resisting $f'_v = \frac{f_v}{2}$.

$$\text{At A, } f'_v = \frac{185}{2}$$
$$= 92.5 \text{ psi.}$$
$$\text{and to the left of C, } f'_v = \frac{176}{2}$$
$$= 88 \text{ psi.}$$

Over the length of beam CB, f_v varies linearly between 88.4 psi. and 106 psi., both values being greater than $f_v = 0.02\ F'_c + 20 = 80$ psi. but less than $f_v = 0.04\ F'_c + 40 = 160$ psi. Therefore only one type of shear reinforcement is necessary, which for a continuous beam must resist $f'_v = f_v - \frac{1}{3}(0.02\ F'_c + 20) = f_v - 26.7$ psi. Select vertical stirrups, to be provided over the full distance CB (assuming that negative reinforcement is continuous across the beam span).

$$\text{At B, } f'_v = 106 - 26.7$$
$$= 79.3 \text{ psi.,}$$
$$\text{and to the right of C, } f'_v = 88.4 - 26.7$$
$$= 61.7 \text{ psi.}$$

(3) Assume $\phi 4$ stirrups.

From Table 13, for $\phi 4$ stirrups, $F_s/F'_c = 20{,}000/3{,}000$ and $d = 40$ in.:

$$\frac{A_{sv}\ F_{sv}}{1{,}000} = 7.85 \text{ kips.}$$

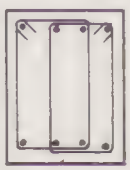
Compute
$$\frac{A_{sv}\ F_{sv}}{1{,}000} \times \frac{B}{b} = 7.85 \times \frac{1.00}{20}$$
$$= 0.393 \text{ kips. per in.}$$

From Diagram 14, for $\frac{A_{sv}\ F_{sv}}{1{,}000} \times \frac{B}{b} = 0.393$, and

	$f'_v = 92.5$ psi. at A :	s	= 4.2 in.,
	$f'_v = 88$ psi. to the left of C :	s	= 4.5 in.,
	$f'_v = 61.7$ psi. to the right of C :	s	= 6.4 in.,
and	$f'_v = 79.3$ psi. at B :	s	= 5.0 in.

Check maximum spacing for vertical stirrups $= \frac{d}{2} = 20$ in. at any point in the beam.

Stirrup spacings of 4 to 6 inches as required would allow sufficient access for placing concrete. Alternatively twice these spacings for single stirrups can be adopted by using double stirrups, arranged, for example, as shown. A suitable practical spacing for $\phi 4$ double stirrups could then be 8 in. between A and C, 10 in. near B and 12 in. to the right of C. The change from 10 in. to 12 in. spacing could be made when, from Diagram 14, and for $\frac{A_{sv}\ F_{sv}}{1{,}000} \times \frac{B}{b} = 0.393$ and $s = \frac{12}{2} = 6.0$ in.:



Double stirrup
= 2 single stirrups
(4 vertical legs)

$$f'_v = 66 \text{ psi.}$$

This value of f'_v occurs at a distance of $13 \times \frac{79.3 - 66}{79.3 - 61.7} = 9.8$ ft. from B.

However, no extra stirrups are required if 10 in. spacing is used throughout BC instead of changing to 12 in. near C. Therefore use 10 in. throughout BC, leaving one odd space of 11 in. as shown opposite.

(4) For inclined bar reinforcement required between A and C only, assume pairs of #8 bars inclined at 45^0 to the neutral axis.

Compute $\frac{V'}{1{,}000jd}$, for $f'_v = 92.5$ psi. at A:

$$\frac{V'}{1{,}000jd} = \frac{f'_v \times b}{1{,}000}$$

$$= \frac{92.5 \times 20}{1{,}000}$$

$$= 1.85 \text{ kips per in.}$$

From Table 15, for $F_{sv} = 20{,}000$ psi., $\theta = 45^0$ and $\frac{V'}{1{,}000jd} = 1.85$:

$s = 12$ in. for a single inclined #8 bar.

For #8 inclined bars in pairs: $s = 24$ in.

At C $$\frac{V'}{1{,}000jd} = \frac{88 \times 20}{1{,}000}$$

$$= 1.76 \text{ kips per in.}$$

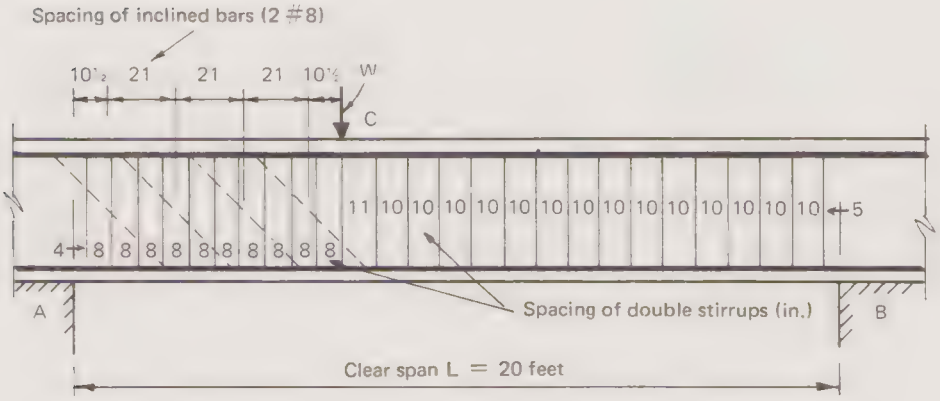
From Table 15 : $s = 12.7$ in. for a single inclined #8 bar,

or for bars in pairs, $s = 25.4$ in.

Check maximum spacing for inclined bars $= \frac{3}{8}(1 + \cot\theta)\, d \leqslant \frac{3}{4} d$

$$= 30 \text{ in.}$$

A possible practical spacing is then 24 in., the midpoint of the inclined section of the first pair of bars being located 12 in. from A. However, the fourth pair of bars is then located 7 feet from A, that is at C. A more satisfactory distribution is obtained without increasing the number of bars, by using a spacing of 21 in. as shown below.


Spacing of inclined bars (2 #8)
10½ 21 21 21 10½ W
C
11 10 10 10 10 10 10 10 10 10 10 10 10 10 5
4 8 8 8 8 8 8 8 8 8 8
A
Spacing of double stirrups (in.)
B
Clear span L = 20 feet

Simple Bending and Bending Combined with Axial Load

The aim in investigation is to determine the actual stresses in concrete and reinforcement for a given section subject to a known loading. The procedure involved is to compute k and j for the section, and consequently to evaluate the stresses. Examples 12 to 16 deal with sections subject to simple bending, and Examples 17 to 23 with bending combined with axial load.

For rectangular sections subject to simple bending, values of k and j can be taken from Tables 10 and 12, respectively, when the quantities of tensile and compressive reinforcement are known. For the purpose of entering Table 10 to obtain k, two quantities m and q must be computed, each of which is a function of the known properties of the section and of the modular ratio. With k known, the value of z which locates the resultant of the compressive forces is obtained from Table 11, and is used in entering Table 12 to determine j. The stresses in the concrete and reinforcement are then computed by the usual equations.

In Example 13 it will be noted that on following the above procedure, the stress in the compressive reinforcement, when transformed at a modular ratio of 2n, exceeds the assumed permissible value F_{sc}. The procedure then followed is to repeat the computation for k, j and the stresses, but using a suitable value of the modular ratio less than 2n so that the computed stress in the compressive reinforcement does not exceed F_{sc}. The reduction in the value of f_{sc} is accompanied by increased values of f_s and f_c which may then be compared with the permissible values F_s and F_c respectively in order to assess the capacity of the section to support the applied moment.

In the investigation of T–sections subject to simple bending when the stress in the stem is neglected, the value of k is computed directly without the use of tables. With k determined, y, defining the centre of gravity of the concrete compressive stresses, is obtained from Table 8. Then j and the stresses are computed (see Example 14).

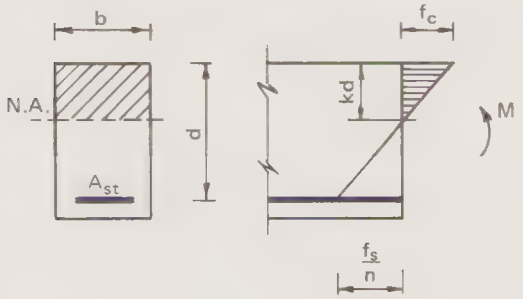
When the stress in the stem of a T–section is taken into account, the problem is similar to that of a rectangular section except that the equations for m, q and j are modified to take into account the dimensions of the flange and the web. Where compressive reinforcement is not used, as in Example 15, the values of z and subsequently of j are taken from Tables 11 and 12, respectively. The introduction of compressive reinforcement requires, however, that j be computed and not taken from the tables (see Example 16). Again, as with the rectangular section, the whole computation may require repetition if the stress in the compressive reinforcement is first found to be excessive. To reduce the amount of arithmetic involved, as demonstrated in Example 16, the possibility that $f_{sc} > F_{sc}$ is determined by establishing that the initially computed value of k is greater than the transition value defined in Table 7. Consequently j is not computed at this stage, but is estimated only, without significantly affecting the accuracy of the finally computed stresses.

The procedure for investigating rectangular and T–sections subject to combined bending and axial load is essentially the same as for the simple bending. It is necessary to make a preliminary estimate of the value of j in order to obtain the value of i, which is subsequently used in computing m and q. A zigzag line in Table 9 serves as a guide in estimating j and i for rectangular or T–sections subject to axial compression. Alternatively, for T–sections in which the stress in the stem is neglected, j may be estimated from Table 8. When the axial force is tensile, i must be computed since Table 9 does not apply. Having computed m and q, the value of k can be taken from Table 10 and then the actual value of j taken from Table 12 or computed, as in the case of simple bending. With the actual value of j, the correct value of i is obtained from Table 9 or by computation, and the stresses then determined.

The above remarks do not apply to eccentrically loaded compression members designed in accordance with Australian Standard CA2–1963, for which investigation procedures are suggested in Examples 28 and 29.

Example 12
Rectangular Section, Simple Bending

In a given rectangular section subject to bending, determine stresses f_s and f_c.



Given:

b = 12 in.
d = 21 in.
A_{st} = 3.98 sq. in.
n = 12
M = 1,330 in. kips.

(1) Compute $$m = q = np = \frac{nA_{st}}{bd} = \frac{12 \times 3.98}{12 \times 21}$$
$$= 0.190.$$

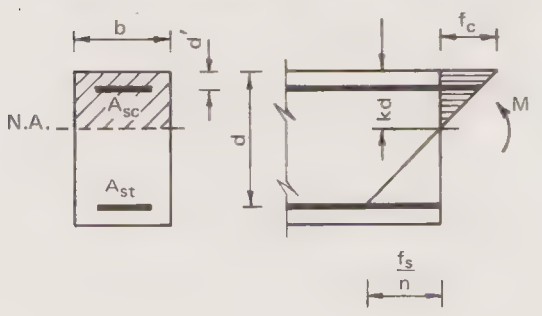
(2) From Table 10, for m = q = 0.190 : $k = 0.455.$
From Table 11, for $A_{sc} = 0$: $z = 0.33.$
From Table 12, for z = 0.33 and k = 0.455: $j = 0.85,$

$$\text{or } j = 1 - \frac{k}{3}$$
$$= 1 - 0.152$$
$$= 0.848.$$

$$\text{Then } f_s = \frac{1{,}000\ M}{j\ d\ A_{st}}$$
$$= \frac{1{,}000 \times 1{,}330}{0.85 \times 21 \times 3.98}$$
$$= 18{,}750 \text{ psi.};$$
$$f_c = \frac{f_s}{n} \times \frac{k}{1-k}$$
$$= \frac{18{,}750}{12} \times \frac{0.455}{0.545}$$
$$= 1{,}300 \text{ psi.}$$

Example 13
Rectangular Section, Simple Bending — Compressive Reinforcement

In a given rectangular section with compressive reinforcement and subject to bending, determine stresses f_s, f_c and f_{sc}.



Given:

b = 9 in.
d = 15 in. d′ = 2 in.
A_{st} = 2.37 sq. in.
A_{sc} = 0.39 sq. in.
n = 12.
M = 600 in. kips.

Example 13 (ctd.)

(1) Compute

$$m = \frac{n A_{st}}{bd} + \frac{(2n-1) A_{sc}}{bd}$$
$$= \frac{12 \times 2.37}{9 \times 15} + \frac{23 \times 0.39}{9 \times 15}$$
$$= 0.211 + 0.066$$
$$= 0.277 ;$$

$$q = np + (2n-1) p' \frac{d'}{d}$$
$$= \frac{nA_{st}}{bd} + \frac{(2n-1) A_{sc}}{bd} \times \frac{d'}{d}$$
$$= 0.211 + 0.066 \times \frac{2}{15}$$
$$= 0.211 + 0.009$$
$$= 0.220.$$

(2) From Table 10, for $m = 0.277$ and $q = 0.220$: $k = 0.442$.
For entering Table 11, determine

$$\frac{1}{k} \times \frac{(2n-1)A_{sc}}{bd} = \frac{0.066}{0.442}$$
$$= 0.150 ;$$
$$\text{and } \frac{1}{k} \times \frac{d'}{d} = \frac{1}{0.442} \times \frac{2}{15}$$
$$= 0.302$$

From Table 11 : $z = 0.33.$

(3) From Table 12, for $z = 0.33$ and $k = 0.442$: $j = 0.85.$

$$\text{Then } f_s = \frac{1{,}000M}{jd\, A_{st}}$$
$$= \frac{1{,}000 \times 600}{0.85 \times 15 \times 2.37}$$
$$= 19{,}900 \text{ psi.};$$
$$f_c = \frac{f_s}{n} \times \frac{k}{1-k}$$
$$= \frac{19{,}900}{12} \times \frac{0.442}{0.558}$$
$$= 1{,}310 \text{ psi.} ;$$
$$f_{sc} = 2f_s \times \frac{k - d'/d}{1-k}$$
$$= 2 \times 19{,}900 \times \frac{0.309}{0.558}$$
$$= 22{,}000 \text{ psi.}$$

If it is assumed that the steel used is of cold worked quality, the computed stress in both the tensile and compressive reinforcement is less than the respective maximum permissible values of $F_s = 30{,}000$ psi. and $F_{sc} = 23{,}000$ psi. For structural grade reinforcement however, $F_s = 20{,}000$ psi. and $F_{sc} = 16{,}500$ psi., so that the compressive reinforcement is significantly overstressed. When the latter condition exists, most designers would deem the section to be satisfactory in terms of its moment carrying capacity, if, by using a suitable modular ratio less than 2n to transform the compressive reinforcement, none of the resultant stresses for concrete and steel exceeds the respective maximum permissible value.* The new modular ratio to be used for the compressive reinforcement may be taken approximately as:

$2n' = 2n \times \frac{F_{sc}}{\text{initially computed value of } f_{sc}}$, although as indicated below there is some advantage to be gained by slightly underestimating this new value.

For the present example, assume structural grade steel and recompute the modular ratio for the compressive reinforcement.

$$2n' = 2 \times 12 \times \frac{16{,}500}{22{,}000}$$

$$= 18.0.$$

Repeat steps (1), (2) and (3).

(4) Compute

$$m = 0.211 + \frac{(18-1) \times 0.39}{9 \times 15}$$

$$= 0.211 + 0.049$$

$$= 0.260;$$

$$\text{and } q = 0.211 + 0.049 \times \frac{2}{15}$$

$$= 0.211 + 0.007$$

$$= 0.218.$$

(5) From Table 10, for m = 0.26 and q = 0.218: $k = 0.449.$

$$\text{Determine } \frac{1}{k} \times \frac{(2n-1)\,A_{sc}}{bd} = \frac{0.049}{0.449}$$

$$= 0.109;$$

$$\text{and } \frac{1}{k} \times \frac{d'}{d} = \frac{1}{0.449} \times \frac{2}{15}$$

$$= 0.297.$$

From Table 11 : $z = 0.33$, as before.

(6) From Table 12, for z = 0.33 and k = 0.449 : $j = 0.85$, as before.

$$\text{Then } f_s = \frac{1{,}000\ M}{j\,d\,A_{st}}$$

$$= \frac{1{,}000 \times 600}{0.85 \times 15 \times 2.37}$$

$$= 19{,}900 \text{ psi.};$$

$$f_c = \frac{f_s}{n} \times \frac{k}{1-k}$$

$$= \frac{19{,}900}{12} \times \frac{0.449}{0.551}$$

$$= 1{,}350 \text{ psi.};$$

$$f_{sc} = \frac{2n'}{n} f_s \times \frac{k - \frac{d'}{d}}{1-k}$$

$$= \frac{18}{12} \times 19{,}900 \times \frac{0.316}{0.551}$$

$$= 17{,}100 \text{ psi.}$$

Note that f_{sc} still exceeds F_{sc} because the value of k, and consequently of $\left[f_s \times \frac{k - \frac{d'}{d}}{1-k}\right]$, increases slightly as the modular ratio for the compressive reinforcement is decreased. If desired the computation could be repeated again to reduce f_{sc} further, with the modular ratio for the compressive reinforcement adjusted to a value of $18.0 \times \frac{16{,}500}{17{,}100} = 17.4$.

Alternatively this iterative procedure could have been avoided by underestimating the modular ratio used in (4) above, to compensate for the resultant increase in k; a value of, say, 17 instead of 18 would have been satisfactory here.

At this stage however a simple and sufficiently accurate procedure is to extrapolate linearly from stress values obtained in (3) and (6):

Then, for $f_{sc} = 16{,}500$ psi. :

Compute

$$f_c = 1{,}350 + (1{,}350 - 1{,}310) \times \frac{17{,}100 - 16{,}500}{22{,}000 - 16{,}500}$$

$$\doteq 1{,}350 \text{ psi.};$$

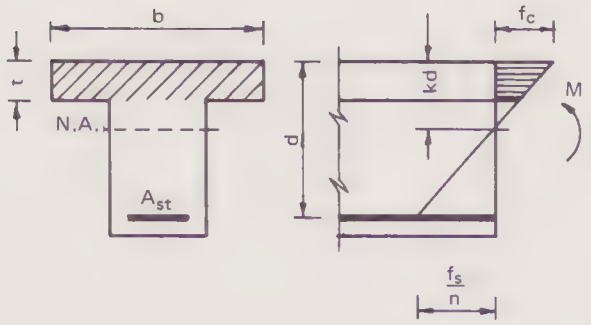
$$\text{and } f_s = 19{,}900 \text{ psi., final.}$$

---

* The technique of reducing the modular ratio for compressive reinforcement to a value less than 2n, in order to limit, when necessary, the computed value of f_{sc} to its permissible value F_{sc}, is employed by the American Concrete Institute in its publication SP–3, "REINFORCED CONCRETE DESIGN HANDBOOK – Working Stress Method", and is followed by many designers in Australia.

Example 14
T–Section, Simple Bending — Stress in Stem Neglected

In a given T–section subject to bending, determine stresses f_s and f_c. Neglect compressive stresses below flange.



Given:
b = 40 in.
d = 25 in.
t = 5 in.
A_{st} = 6.96 sq. in.
n = 12.
M = 3,000 in. kips.

(1) Compute

$$k = \frac{\frac{n A_{st}}{bt} + \frac{1}{2} \times \frac{t}{d}}{\frac{n A_{st}}{bt} + 1}$$

$$= \frac{\frac{12 \times 6.96}{40 \times 5} + \frac{1}{2} \times \frac{5}{25}}{\frac{12 \times 6.96}{40 \times 5} + 1.000}$$

$$= \frac{0.418}{0.418} + \frac{0.100}{1.000}$$

$$= \frac{0.518}{1.418}$$

$$= 0.366.$$

Check position of neutral axis : $kd = 0.366 \times 25 = 9.1$ in. $> t$.
Therefore neutral axis lies below flange.

(2) Compute

$$\frac{t}{kd} = \frac{5}{0.366 \times 25}$$

$$= 0.547.$$

From Table 8, for $\frac{t}{kd} = 0.55$: $y = 0.44$.

Then $j = 1 - y\frac{t}{d}$

$$= 1 - 0.44 \times \frac{5}{25}$$

$$= 1 - 0.088$$

$$= 0.912.$$

Then $f_s = \frac{1{,}000\ M}{j\ d\ A_{st}}$

$$= \frac{1{,}000 \times 3{,}000}{0.912 \times 25 \times 6.96}$$

$$= 18{,}900 \text{ psi.};$$

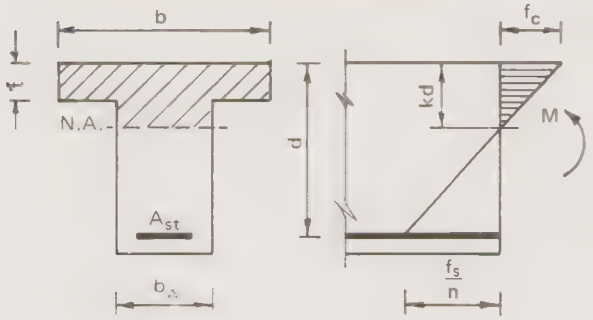
$$f_c = \frac{f_s}{n} \times \frac{k}{1-k}$$

$$= \frac{18{,}900}{12} \times \frac{0.366}{0.634}$$

$$= 910 \text{ psi.}$$

Example 15
T–Section, Simple Bending – Stress in Stem Included

In a given T–section with all concrete in compression considered effective and subject to bending, determine stresses f_s and f_c.



Given:

$b = 40$ in. $b_w = 15$ in.
$d = 25$ in.
$t = 5$ in.
$A_{st} = 6.96$ sq. in.
$n = 12$.
$M = 3{,}000$ in. kips.

(1) Compute

$$m = np + p'' = \frac{nA_{st}}{b_w d} + \frac{(b - b_w)t}{b_w d}$$

$$= \frac{12 \times 6.96}{15 \times 25} + \frac{(40 - 15) \times 5}{15 \times 25}$$

$$= 0.223 + 0.333$$

$$= 0.556;$$

$$q = np + p''\frac{t}{2d} = \frac{nA_{st}}{b_w d} + \frac{(b - b_w)t}{b_w d} \times \frac{t}{2d}$$

$$= 0.223 + 0.333 \times \frac{5}{2 \times 25}$$

$$= 0.223 + 0.033$$

$$= 0.256.$$

(2) From Table 10, for $m = 0.556$ and $q = 0.256$: $k = 0.350$.

Check position of neutral axis $kd = 0.350 \times 25 = 8.8$ in. $> t$.
Therefore neutral axis lies below flange.

Compute $\frac{t}{kd} = \frac{5}{0.350 \times 25}$

$$= 0.57.$$

From Table 8, for $\frac{t}{kd} = 0.57$: $y = 0.435$.

For entering Table 11, compute $\frac{1}{k} \times \frac{(b - b_w)t}{b_w d} = \frac{0.333}{0.350}$

$$= 0.95;$$

and $\frac{1}{k} \times \frac{yt}{d} = \frac{0.435 \times 5}{0.350 \times 25}$

$$= 0.25.$$

From Table 11 : $z = 0.285$

(3) From Table 12, for $z = 0.285$ and $k = 0.350$: $j = 0.90$.

Then $f_s = \frac{1{,}000\ M}{j\ d\ A_{st}} = \frac{1{,}000 \times 3{,}000}{0.90 \times 25 \times 6.96}$

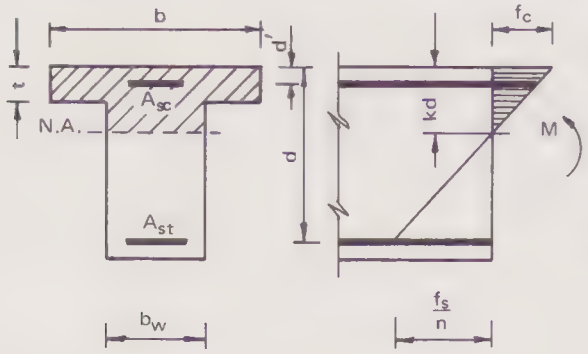
$$= 19{,}200 \text{ psi.};$$

$$f_c = \frac{f_s}{n} \times \frac{k}{1 - k} = \frac{19{,}200}{12} \times \frac{0.350}{0.650}$$

$$= 860 \text{ psi.}$$

Example 16
T–Section, Simple Bending – Stress in Stem Included, Compressive Reinforcement

In a given T–section with compressive reinforcement and subject to bending, determine stresses f_s, f_c and f_{sc}. Include compressive stresses below flange.



Given:

b = 35 in. b_w = 18 in.
d = 22 in. d′ = 2 in.
t = 5 in.
A_{st} = 10.9 sq. in.
A_{sc} = 0.88 sq. in.
n = 12.
M = 4,200 in. kips.

(1) Compute

$$m = np + (2n-1)\,p' + p''$$

$$= \frac{nA_{st}}{b_w d} + \frac{(2n-1)A_{sc}}{b_w d} + \frac{(b-b_w)t}{b_w d}$$

$$= \frac{12 \times 10.9}{18 \times 22} + \frac{23 \times 0.88}{18 \times 22} + \frac{(35-18) \times 5}{18 \times 22}$$

$$= 0.330 + 0.051 + 0.215$$

$$= 0.596;$$

$$q = np + (2n-1)p'\frac{d'}{d} + p''\frac{t}{2d}$$

$$= \frac{nA_{st}}{b_w d} + \frac{(2n-1)A_{sc}}{b_w d} \times \frac{d'}{d} + \frac{(b-b_w)t}{b_w d} \times \frac{t}{2d}$$

$$= 0.330 + 0.051 \times \frac{2}{22} + 0.215 \times \frac{5}{2 \times 22}$$

$$= 0.330 + 0.005 + 0.024$$

$$= 0.359.$$

(2) From Table 10, for m = 0.596 and q = 0.359 : k = 0.440.

Check position of neutral axis : kd = 0.440 x 22 = 9.7 in > t.
Therefore neutral axis lies below flange.

At this stage the normal procedure is to evaluate j and the stresses. If f_{sc} is found to exceed the maximum permissible value F_{sc}, however, the computation may have to be repeated, using a modular ratio of less than 2n to transform the compressive reinforcement, before a final decision is taken as to whether or not the section is satisfactory.* When repetition is necessary, it is convenient and sufficiently accurate to estimate j during the first part of the computation, the precise evaluation being made only during the second cycle.
The possibility that $f_{sc} > F_{sc}$ can be established by computing the transition value of k (see Table 7) as follows:

For F_s/F_{sc} = 20,000/16,500, assumed : transition $k = 0.292 + 0.708\frac{d'}{d}$

$$= 0.292 + 0.708 \times \frac{2}{22}$$

$$= 0.356.$$

* See Example 13

Since k = 0.440 exceeds the transition value, it is possible (but not necessary)* that the stress in the compressive reinforcement exceeds F_{sc} = 16,500 psi. To estimate whether this is so, assume a value of j and compute the stresses:

$$\text{Assume } j = 0.90.$$

$$\text{Estimate } f_s = \frac{1{,}000\ M}{j\ d\ A_{st}} = \frac{1{,}000 \times 4{,}200}{0.90 \times 22 \times 10.9} = 19{,}500 \text{ psi.;}$$

$$f_c = \frac{f_s}{n} \times \frac{k}{1-k} = \frac{19{,}500}{12} \times \frac{0.440}{0.560} = 1{,}280 \text{ psi.;}$$

$$f_{sc} = 2f_s \times \frac{k - \frac{d'}{d}}{1-k} = 2 \times 19{,}500 \times \frac{(0.440 - 0.091)}{0.560} = 24{,}300 \text{ psi.}$$

Because f_{sc} considerably exceeds F_{sc}, recompute k, j and the stresses using a modular ratio for the compressive reinforcement based upon a value of:

$$2n' = 2n \times \frac{F_{sc}}{\text{initially computed value of } f_{sc}} = 2 \times 12 \times \frac{16{,}500}{24{,}300} = 16.3.$$

This ratio is further underestimated to a value of say $2n' = 15.0$ to eliminate the need for more than one repetition of the computation.** Then $(2n - 1)$ in the equations for m, q and j becomes $2n' - 1 = 15 - 1 = 14$.

(3) Recalculate

$$m = 0.330 + \frac{14 \times 0.88}{18 \times 22} + 0.215 = 0.330 + 0.031 + 0.215 = 0.576;$$

$$q = 0.330 + 0.031 \times \frac{2}{22} + 0.024 = 0.330 + 0.003 + 0.024 = 0.357.$$

(4) From Table 10, for m = 0.576 and q = 0.357 : k = 0.447.

(5) The general equation for j is :

$$j = 1 - \frac{\frac{k^2}{6} + \frac{(2n-1)A_{sc}}{b_w d} \times \frac{d'}{d} \times (1 - \frac{d'}{kd}) + \frac{(b - b_w)t}{b_w d} \times \frac{yt}{d} \times (1 - \frac{t}{2kd})}{\frac{k}{2} + \frac{(2n-1)A_{sc}}{b_w d}(1 - \frac{d'}{kd}) + \frac{(b - b_w)t}{b_w d}(1 - \frac{t}{2kd})}.$$

* Even though k exceeds the transition value, f_{sc} may be less than F_{sc}, if f_s is also small by comparison with F_s (see Example 18).

** See Example 13.

Example 16 (ctd.)

Compute

$$1-\frac{d'}{kd} = 1-\frac{2}{0.447 \times 22}$$
$$= 1-0.203$$
$$= 0.797,$$
$$\text{and } \frac{t}{kd} = \frac{5}{0.447 \times 22}$$
$$= 0.508.$$

From Table 8, for $\frac{t}{kd} = 0.51$: $y = 0.44.$

$$\text{Then } \frac{yt}{d} = \frac{0.44 \times 5}{22}$$
$$= 0.100,$$
$$\text{and } 1-\frac{t}{2kd} = 1-\frac{0.508}{2}$$
$$= 0.746.$$

$$\text{Then } j = 1-\frac{\frac{0.447^2}{6} + 0.003 \times 0.797 + 0.215 \times 0.100 \times 0.746}{\frac{0.447}{2} + 0.031 \times 0.797 + 0.215 \times 0.746}$$
$$= 1-\frac{0.033 + 0.002 + 0.016}{0.224 + 0.025 + 0.160}$$
$$= 1-\frac{0.051}{0.409}$$
$$= 0.875.$$

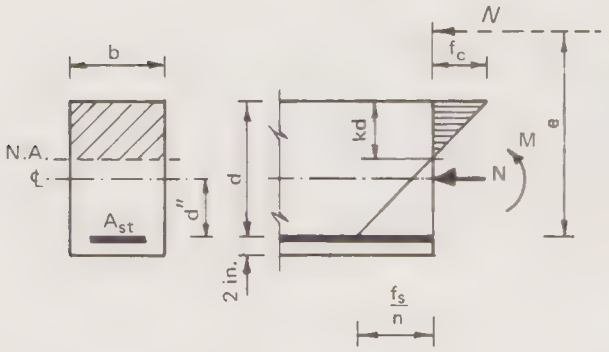
(6) Compute

$$f_s = \frac{1{,}000\ M}{j\ d\ A_{st}}$$
$$= \frac{1{,}000 \times 4{,}200}{0.875 \times 22 \times 10.9}$$
$$= 20{,}000 \text{ psi.};$$
$$f_c = \frac{f_s}{n} \times \frac{k}{1-k}$$
$$= \frac{20{,}000}{12} \times \frac{0.447}{0.553}$$
$$= 1{,}350 \text{ psi.};$$
$$f_{sc} = \frac{2n'}{n} f_s \times \frac{k-\frac{d'}{d}}{1-k}$$
$$= \frac{15}{12} \times 20{,}000 \times \frac{0.447-0.091}{0.553}$$
$$= 16{,}100 \text{ psi.}$$

Note that f_{sc} is now slightly less than F_{sc}, because the revised modular ratio was underestimated by too much. However since none of the computed stresses now exceeds its permissible value, the section may be regarded as satisfactorily supporting the applied moment M, and further adjustment to f_{sc} is unnecessary.

Example 17
Rectangular Section, Combined Bending and Axial Load

In a given rectangular section subject to bending moment M combined with axial load N, determine stresses f_s and f_c.



Given:

$b = 12$ in.
$d = 22$ in. $d'' = 10$ in.
$A_{st} = 2.98$ sq. in.
$n = 12$.
$M = 1{,}290$ in. kips.
$N = 30$ kips.

(1) Compute

$$e = \frac{M}{N} + d''$$
$$= \frac{1{,}290}{30} + 10$$
$$= 53 \text{ in.},$$
$$\text{and } \frac{e}{d} = \frac{53}{22}$$
$$= 2.41.$$

(2) From Table 9, for $\frac{e}{d} = 2.41$ and $j = 0.87$ (estimated) : $i = 1.57$.

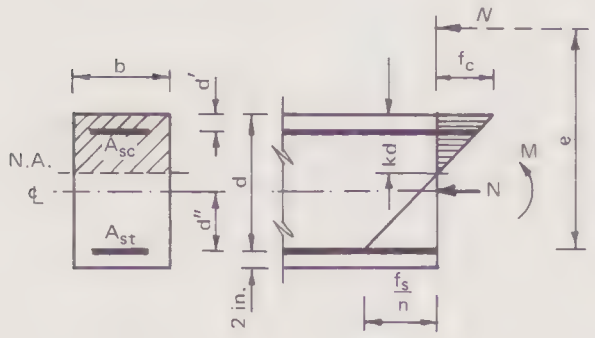
$$\text{Then } m = q = \frac{n A_{st} i}{bd}$$
$$= \frac{12 \times 2.98 \times 1.57}{12 \times 22}$$
$$= 0.213.$$

(3) From Table 10, for $m = q = 0.213$: $k = 0.473$.
From Table 11, for $A_{sc} = 0$: $z = 0.33$.
From Table 12, for $z = 0.33$ and $k = 0.473$: $j = 0.84$ (actual).
From Table 9, for $\frac{e}{d} = 2.41$ and $j = 0.84$: $i = 1.54$.

$$\text{Then } f_s = \frac{1{,}000\ N}{j\ A_{st}\ i} \times \frac{e}{d}$$
$$= \frac{1{,}000 \times 30}{0.84 \times 2.98 \times 1.54} \times 2.41$$
$$= 18{,}700 \text{ psi.};$$
$$f_c = \frac{f_s}{n} \times \frac{k}{1 - k}$$
$$= \frac{18{,}700}{12} \times \frac{0.473}{0.527}$$
$$= 1{,}400 \text{ psi.}$$

Example 18
Rectangular Section, Combined Bending and Axial Load – Compressive Reinforcement

In a given section with compressive reinforcement and subject to bending moment M combined with axial load N, determine stresses f_s, f_c and f_{sc}.



Given:

$b = 12$ in. $d' = 2$ in.
$d = 22$ in. $d'' = 10$ in.
$A_{st} = 2.98$ sq. in.
$A_{sc} = 0.39$ sq. in.
$n = 12$.
$M = 860$ in. kips.
$N = 20$ kips.

(1) Compute

$$e = \frac{M}{N} + d''$$

$$= \frac{860}{20} + 10$$

$$= 53 \text{ in.},$$

$$\text{and } \frac{e}{d} = \frac{53}{22}$$

$$= 2.41.$$

(2) From Table 9, for $\frac{e}{d} = 2.41$ and $j = 0.87$ (estimated) : $i = 1.57$.

$$\text{Then } m = npi + (2n-1)p'$$

$$= \frac{nA_{st}i}{bd} + \frac{(2n-1)\,A_{sc}}{bd}$$

$$= \frac{12 \times 2.98 \times 1.57}{12 \times 22} + \frac{23 \times 0.39}{12 \times 22}$$

$$= 0.213 + 0.034$$

$$= 0.247 ;$$

$$q = npi + (2n-1)p'\frac{d'}{d}$$

$$= \frac{nA_{st}i}{bd} + \frac{(2n-1)A_{sc}}{bd} \times \frac{d'}{d}$$

$$= 0.213 + 0.034 \times \frac{2}{22}$$

$$= 0.213 + 0.003$$

$$= 0.216.$$

(3)* From Table 10, for $m = 0.247$ and $q = 0.216$: $k = 0.454$.
Compute transition value for k, assuming $F_s/F_{sc} = 20{,}000/16{,}500$.

$$\text{transition k (see Table 7)} = 0.292 + 0.708 \times \frac{d'}{d}$$

$$= 0.292 + 0.708 \times 0.091$$

$$= 0.292 + 0.064$$

$$= 0.356.$$

Since $k = 0.454$ exceeds the transition value it is possible (but not necessary)** that the stress in the compressive reinforcement exceeds its maximum permissible value of $F_{sc} = 16{,}500$ psi. (structural grade steel). The value of f_{sc} may be quickly estimated, using the assumed value of j, as follows:

---

* The reason for the procedure adopted in Section (3) above, in which the transition value of k is computed and j and hence f_{sc} estimated, is given in Section (2) of Example 16.

** Even though k exceeds the transition value, f_{sc} may be less than F_{sc}, if f_s is also small by comparison with F_s, as subsequently occurs in this example.

$$\text{Estimate } f_s = \frac{1{,}000\ N}{j\ A_{st}\ i} \times \frac{e}{d}$$

$$= \frac{1{,}000 \times 20}{0.87 \times 2.98 \times 1.57} \times 2.41$$

$$= 11{,}800 \text{ psi.};$$

$$f_{sc} = 2\ f_s \times \frac{k - \frac{d'}{d}}{1 - k}$$

$$= 2 \times 11{,}800 \times \frac{0.363}{0.546}$$

$$= 15{,}700 \text{ psi.}$$

Since estimated $f_{sc} < F_{sc}$, proceed with the precise determination of j and the stresses.

(4) For entering Table 11, compute $\frac{1}{k} \times \frac{(2n-1)A_{sc}}{bd} = \frac{0.034}{0.454}$

$$= 0.075,$$

$$\text{and } \frac{1}{k} \times \frac{d'}{d} = \frac{0.091}{0.454}$$

$$= 0.200.$$

From Table 11 : $z = 0.32.$

(5) From Table 12, for $z = 0.32$ and $k = 0.454$: $j = 0.85$ (actual).

From Table 9, for $\frac{e}{d} = 2.41$ and $j = 0.85$: $i = 1.55.$

$$\text{Then } f_s = \frac{1{,}000\ N}{j\ A_{st}\ i} \times \frac{e}{d}$$

$$= \frac{1{,}000 \times 20}{0.85 \times 2.98 \times 1.55} \times 2.41$$

$$= 12{,}300 \text{ psi.};$$

$$f_c = \frac{f_s}{n} \times \frac{k}{1 - k}$$

$$= \frac{12{,}300}{12} \times \frac{0.454}{0.546}$$

$$= 850 \text{ psi.};$$

$$f_{sc} = 2\ f_s \times \frac{k - \frac{d'}{d}}{1 - k}$$

$$= 2 \times 12{,}300 \times \frac{0.363}{0.546}$$

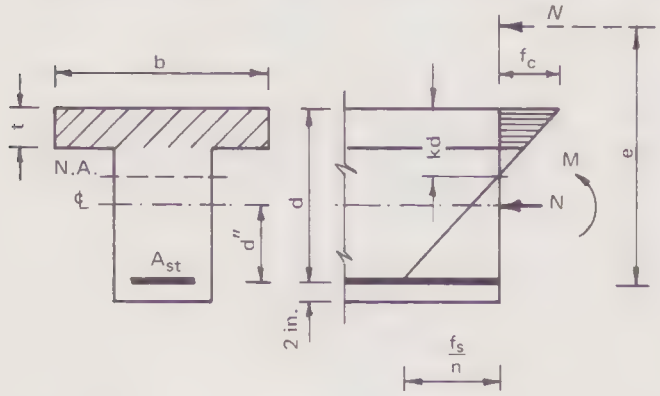
$$= 16{,}400 \text{ psi.} *$$

---

* f_{sc} is less than F_{sc} only because f_s is much less than F_s.

Example 19
T–Section, Combined Bending and Axial Load – Stress in Stem Neglected

In a given T–section subject to bending moment M combined with axial load N, determine stresses f_s and f_c. Neglect compressive stresses below flange.



Given:
b = 30 in.
d = 21 in. d" = 9.5 in.
t = 4 in.
A_{st} = 2.36 sq. in.
n = 12.
M = 1,200 in. kips.
N = 30 kips.

(1) Compute

$$e = \frac{M}{N} + d''$$
$$= \frac{1{,}200}{30} + 9.5$$
$$= 49.5 \text{ in.},$$
$$\text{and } \frac{e}{d} = \frac{49.5}{21}$$
$$= 2.36.$$

(2) From Table 8, for $\frac{t}{d} = \frac{4}{21} = 0.190$: $j = 0.91$ (estimated by assuming an F_c and F_s).

From Table 9, for $\frac{e}{d} = 2.36$ and $j = 0.91$: $i = 1.63$.

$$\text{Then } k = \frac{\frac{nA_{st}\, i}{bt} + \frac{1}{2} \times \frac{t}{d}}{\frac{nA_{st}\, i}{bt} + 1}$$
$$= \frac{\frac{12 \times 2.36 \times 1.63}{30 \times 4} + \frac{1}{2} \times 0.190}{\frac{12 \times 2.36 \times 1.63}{30 \times 4} + 1.000}$$
$$= \frac{0.385 + 0.095}{0.385 + 1.000}$$
$$= \frac{0.480}{1.385}$$
$$= 0.346.$$
$$\text{Check kd} = 0.346 \times 21$$
$$= 7.3 \text{ in.} > t.$$

Therefore neutral axis lies below flange.

(3) Compute

$$\frac{t}{kd} = \frac{0.190}{0.346}$$
$$= 0.549.$$

From Table 8, for $\frac{t}{kd} = 0.55$: $y = 0.44$.

$$\text{Then } j = 1 - y\frac{t}{d}$$
$$= 1 - 0.44 \times 0.190$$
$$= 1 - 0.084$$
$$= 0.916 \text{ (actual)}.$$

From Table 9, for $\frac{e}{d} = 2.36$ and $j = 0.916$: $i = 1.64$.

$$\text{Then } f_s = \frac{1{,}000\ N}{j\ A_{st}\ i} \times \frac{e}{d}$$

$$= \frac{1{,}000 \times 30}{0.916 \times 2.36 \times 1.64} \times 2.36$$

$$= 20{,}000 \text{ psi. ;}$$

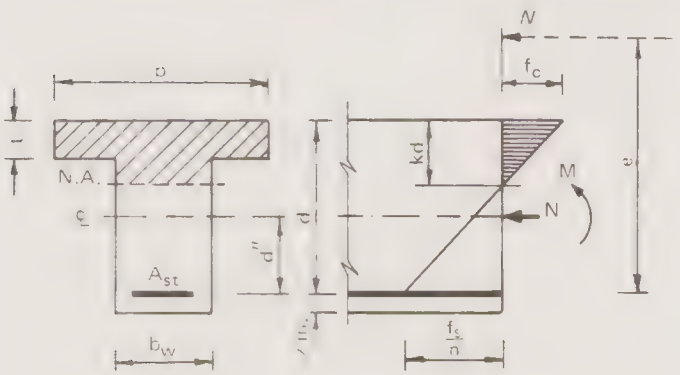
$$f_c = \frac{f_s}{n} \times \frac{k}{1-k}$$

$$= \frac{20{,}000}{12} \times \frac{0.346}{0.654}$$

$$= 880 \text{ psi.}$$

Example 20
T–Section, Combined Bending and Axial Load – Stress in Stem Included

In a given T–section subject to bending moment M combined with axial load N, determine stresses f_s and f_c. Include compressive stresses below flange.



Given:

b = 30 in. b_w = 12 in.
d = 21 in. d'' = 9.5 in.
t = 4 in.
A_{st} = 2.36 sq. in.
n = 12.
M = 1,200 in. kips.
N = 30 kips.

(1) Compute

$$e = \frac{M}{N} + d''$$

$$= \frac{1{,}200}{30} + 9.5$$

$$= 49.5 \text{ in.,}$$

$$\text{and } \frac{e}{d} = \frac{49.5}{21}$$

$$= 2.36.$$

(2) From Table 9, for $\frac{e}{d} = 2.36$ and $j = 0.87$ (estimated) : $i = 1.59$.

$$\text{Then } m = \frac{nA_{st}\ i}{b_w d} + \frac{(b - b_w)t}{b_w d}$$

$$= \frac{12 \times 2.36 \times 1.59}{12 \times 21} + \frac{(30 - 12) \times 4}{12 \times 21}$$

$$= 0.179 + 0.286$$

$$= 0.465 ;$$

$$q = \frac{nA_{st}\ i}{b_w d} + \frac{(b - b_w)t}{b_w d} \times \frac{t}{2d}$$

$$= 0.179 + 0.286 \times \frac{4}{2 \times 21}$$

$$= 0.179 + 0.027$$

$$= 0.206.$$

Example 20 (ctd.)

(3) From Table 10, for m = 0.465 and q = 0.206 : k = 0.328

$$\text{Check kd} = 0.328 \times 21$$
$$= 6.9 \text{ in.} > t.$$

Therefore neutral axis lies below flange.

$$\text{Compute } \frac{t}{kd} = \frac{4}{0.328 \times 21}$$
$$= 0.58.$$

From Table 8, for $\frac{t}{kd} = 0.58$: $y = 0.43.$

For entering Table 11, compute $\frac{1}{k} \times \frac{(b - b_w)t}{b_w d} = \frac{0.286}{0.328}$

$$= 0.87,$$

$$\text{and } \frac{1}{k} \times \frac{yt}{d} = \frac{0.43 \times 4}{0.328 \times 21}$$
$$= 0.25.$$

From Table 11: $z = 0.285.$

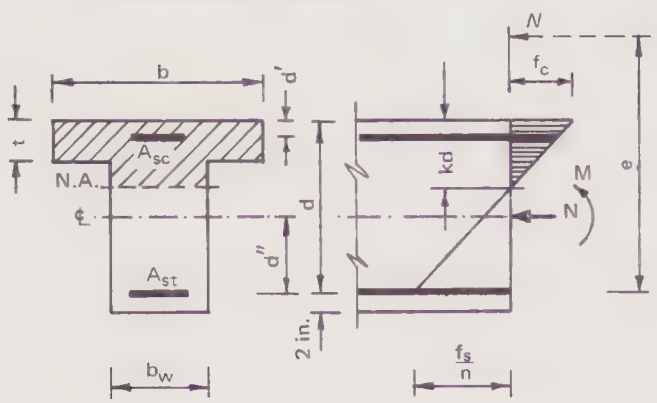
(4) From Table 12, for z = 0.285 and k = 0.328 : j = 0.905 (actual).

From Table 9, for $\frac{e}{d} = 2.36$ and j = 0.905 : i = 1.62.

$$\text{Then } f_s = \frac{1{,}000\ N}{j\ A_{st}\ i} \times \frac{e}{d}$$
$$= \frac{1{,}000 \times 30}{0.905 \times 2.36 \times 1.62} \times 2.36$$
$$= 20{,}500 \text{ psi.};$$
$$f_c = \frac{f_s}{n} \times \frac{k}{1 - k}$$
$$= \frac{20{,}500}{12} \times \frac{0.328}{0.672}$$
$$= 830 \text{ psi.}$$

Example 21
T–Section, Combined Bending and Axial Load – Stress in Stem Included, Compressive Reinforcement

In a given T–section with compressive reinforcement and subject to bending moment M combined with axial load N, determine stresses f_s, f_c and f_{sc}. Include compressive stresses below flange.



Given:

b	= 30 in.	b_w	= 10.5 in.
d	= 21 in.	d''	= 9.5 in.
t	= 5 in.	d'	= 2 in.
A_{st}	= 3.40 sq. in.		
A_{sc}	= 1.99 sq. in.		
n	= 12.		
M	= 1,600 in. kips.		
N	= 40 kips.		

(1) Compute

$$e = \frac{M}{N} + d''$$

$$= \frac{1{,}600}{40} + 9.5$$

$$= 49.5 \text{ in.},$$

$$\text{and } \frac{e}{d} = \frac{49.5}{21}$$

$$= 2.36.$$

(2) From Table 9, for $\frac{e}{d} = 2.36$ and $j = 0.87$ (estimated) : $i = 1.59$.

$$\text{Then } m = \frac{nA_{st}i}{b_w d} + \frac{(2n-1)A_{sc}}{b_w d} + \frac{(b-b_w)t}{b_w d}$$

$$= \frac{12 \times 3.40 \times 1.59}{10.5 \times 21} + \frac{23 \times 1.99}{10.5 \times 21} + \frac{(30-10.5) \times 5}{10.5 \times 21}$$

$$= 0.294 + 0.208 + 0.442$$

$$= 0.944 ;$$

$$q = \frac{nA_{st}i}{b_w d} + \frac{(2n-1)A_{sc}}{b_w d} \times \frac{d'}{d} + \frac{(b-b_w)t}{b_w d} \times \frac{t}{2d}$$

$$= 0.294 + 0.208 \times \frac{2}{21} + 0.442 \times \frac{5}{2 \times 21}$$

$$= 0.294 + 0.020 + 0.053$$

$$= 0.367.$$

(3) From Table 10, for $m = 0.944$ and $q = 0.367$: $k = 0.331$.

$$\text{Check } kd = 0.331 \times 21$$

$$= 7.0 \text{ in.} > t$$

Therefore neutral axis lies below flange.

Compute transition value for k, assuming $F_s/F_{sc} = 20{,}000/16{,}500$:

$$\text{transition k (see Table 7)} = 0.292 + 0.708 \frac{d'}{d}$$

$$= 0.292 + 0.708 \times \frac{2}{21}$$

$$= 0.292 + 0.067$$

$$= 0.359.$$

Since $k = 0.331$ is less than the transition value, the compressive reinforcement will be over-stressed only if the tensile reinforcement is also over-stressed. Therefore j is evaluated accurately at this point.*

(4) The general equation for j is

$$j = 1 - \frac{\frac{k^2}{6} + \frac{(2n-1)A_{sc}}{b_w d} \times \frac{d'}{d} \times \left(1 - \frac{d'}{kd}\right) + \frac{(b-b_w)t}{b_w d} \times \frac{yt}{d} \times \left(1 - \frac{1}{2kd}\right)}{\frac{k}{2} + \frac{(2n-1)A_{sc}}{b_w d}\left(1 - \frac{d'}{kd}\right) + \frac{(b-b_w)t}{b_w d}\left(1 - \frac{t}{2kd}\right)}.$$

* Refer to Examples 13, 16 and 18 for procedure if k is found to exceed the transition value.

Compute $1-\frac{d'}{kd} = 1-\frac{2}{0.331 \times 21}$

$= 1-0.287$

$= 0.713$

and $\frac{t}{kd} = \frac{5}{0.331 \times 21}$

$= 0.719.$

From Table 8, for $\frac{t}{kd} = 0.72$: $y = 0.40.$

Then $\frac{yt}{d} = \frac{0.40 \times 5}{21}$

$= 0.095,$

and $1-\frac{t}{2kd} = 1-0.5 \times 0.719$

$= 0.640.$

Then $j = 1-\frac{\frac{(0.331)^2}{6} + 0.020 \times 0.713 + 0.442 \times 0.095 \times 0.640}{\frac{0.331}{2} + 0.208 \times 0.713 + 0.442 \times 0.640}$

$= 1-\frac{0.018 + 0.014 + 0.027}{0.166 + 0.148 + 0.283}$

$= 1-\frac{0.059}{0.597}$

$= 0.901$ (actual).

(5) From Table 9, for $\frac{e}{d} = 2.36$ and $j = 0.901$: $i = 1.62.$

Then $f_s = \frac{1{,}000\ N}{j\ A_{st}\ i} \times \frac{e}{d}$

$= \frac{1{,}000 \times 40}{0.901 \times 3.40 \times 1.62} \times 2.36$

$= 19{,}000$ psi. ;

$f_c = \frac{f_s}{n} \times \frac{k}{1-k}$

$= \frac{19{,}000}{12} \times \frac{0.331}{0.669}$

$= 780$ psi. ;

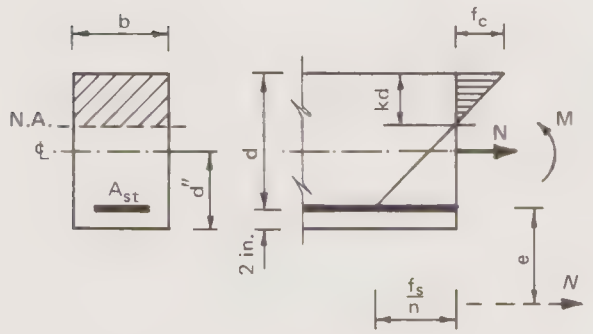
$f_{sc} = 2f_s \times \frac{k-d'/d}{1-k}$

$= 2 \times 19{,}000 \times \frac{0.236}{0.669}$

$= 13{,}400$ psi.

Example 22
Rectangular Section, Combined Bending and Axial Load (Tension)

In a given rectangular section subject to bending moment M combined with axial tension N, determine stresses f_s and f_c.



Given:
b = 14 in;
d = 22 in. d" = 10 in.
A_{st} = 8.40 sq. in.
n = 12.
M = 2,740 in. kips.
N = −45 kips (negative because axial tension).

(1) Compute

$$e = \frac{M}{N} + d''$$

$$= \frac{2{,}740}{-45} + 10$$

$$= -50.9 \text{ in.},$$

$$\text{and } \frac{e}{d} = \frac{-50.9}{22}$$

$$= -2.31.$$

(2) Estimate j = 0.90 and determine

$$i = \frac{1}{1 - \frac{jd}{e}}$$

$$= \frac{1}{1 - \frac{0.90}{-2.31}}$$

$$= 0.72.$$

$$\text{Then } m = q = \frac{nA_{st}\, i}{bd}$$

$$= \frac{12 \times 8.40 \times 0.72}{14 \times 22}$$

$$= 0.236.$$

(3) From Table 10, for m = q = 0.236 : k = 0.490.
From Table 12, for z = 0.33 and k = 0.490 : j = 0.84 (actual).

For j = 0.84, determine i

$$= \frac{1}{1 - \frac{0.84}{-2.31}}$$

$$= 0.73.$$

$$\text{Then } f_s = \frac{1{,}000\, N}{j\, A_{st}\, i} \times \frac{e}{d}$$

$$= \frac{1{,}000 \times (-45)}{0.84 \times 8.40 \times 0.73} \times (-2.31)$$

$$= 20{,}200 \text{ psi.};$$

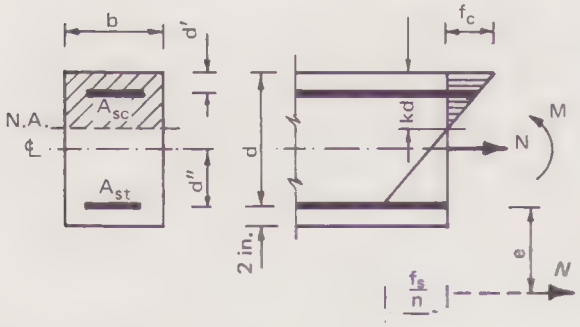
$$f_c = \frac{f_s}{n} \times \frac{k}{1-k}$$

$$= \frac{20{,}200}{12} \times \frac{0.490}{0.510}$$

$$= 1{,}620 \text{ psi.}$$

Example 23
Rectangular Section, Combined Bending and Axial Load (Tension) – Compressive Reinforcement

In a given rectangular section with compressive reinforcement and subject to bending moment M combined with axial tension N, determine stresses f_s, f_c and f_{sc}.



Given:

b = 14 in. d′ = 2 in.
d = 22 in. d″ = 10 in.
A_{st} = 8.40 sq. in.
A_{sc} = 1.80 sq. in.
n = 12.
M = 2,740 in. kips.
N = −45 kips (negative because axial tension).

(1) Compute

$$e = \frac{M}{N} + d''$$

$$= \frac{2{,}740}{-45} + 10$$

$$= -50.9 \text{ in.},$$

$$\text{and } \frac{e}{d} = \frac{-50.9}{22}$$

$$= -2.31.$$

(2) Estimate j = 0.90 and determine

$$i = \frac{1}{1 - \frac{id}{e}}$$

$$= \frac{1}{1 - \frac{0.90}{-2.31}}$$

$$= 0.72.$$

$$\text{Then } m = \frac{nA_{st}\, i}{bd} + \frac{(2n-1)A_{sc}}{bd}$$

$$= \frac{12 \times 8.40 \times 0.72}{14 \times 22} + \frac{23 \times 1.80}{14 \times 22}$$

$$= 0.236 + 0.134$$

$$= 0.370;$$

$$q = \frac{nA_{st}\, i}{bd} + \frac{(2n-1)A_{sc}}{bd} \times \frac{d'}{d}$$

$$= 0.236 + 0.134 \times \frac{2}{22}$$

$$= 0.236 + 0.012$$

$$= 0.248.$$

(3)* From Table 10, for m = 0.370 and q = 0.248 : k = 0.426.

Compute transition value for k, assuming F_s/F_{sc} = 20,000/16,500 :

$$\text{transition k (see Table 7)} = 0.292 + 0.708 \times \frac{d'}{d}$$

$$= 0.292 + 0.708 \times 0.091$$

$$= 0.356.$$

The reason for the procedure adopted in Section (3) above, in which the transition value of k is computed, and j and hence f_{sc} estimated, is given in Section (2) of Example 16.

Since k = 0.426 exceeds the transition value it is possible (but not necessary)* that the stress in the compressive reinforcement exceeds its maximum permissible value of F_{sc} = 16,500 psi. (structural grade steel). The value of f_{sc} may be estimated, using the assumed value of j, as follows:

$$\text{Estimate } f_s = \frac{1{,}000\ N}{j\ A_{st}\ i} \times \frac{e}{d}$$

$$= \frac{1{,}000 \times (-45)}{0.90 \times 8.40 \times 0.72} \times (-2.31)$$

$$= 19{,}100 \text{ psi.};$$

$$f_c = \frac{f_s}{n} \times \frac{k}{1-k}$$

$$= \frac{19{,}100}{12} \times \frac{0.426}{0.574}$$

$$= 1{,}180 \text{ psi.};$$

$$f_{sc} = 2\, f_s \times \frac{k - \frac{d'}{d}}{1-k}$$

$$= 2 \times 19{,}100 \times \frac{0.335}{0.574}$$

$$= 22{,}300 \text{ psi.} > F_{sc}.$$

Therefore recompute the modular ratio for the compressive reinforcement as

$$2n \times \frac{F_{sc}}{f_{sc}} = 2 \times 12 \times \frac{16{,}500}{22{,}300}$$

$$= 17.8.$$

To avoid the necessity for repeating the computation more than once, underestimate the ratio further to a value of, say, $2n' = 15.8$, so that $(2n-1)$ becomes $(2n'-1) = 15.8-1 = 14.8$.

(4) Recalculate

$$m = 0.236 + \frac{14.8 \times 1.80}{14 \times 22}$$

$$= 0.236 + 0.086$$

$$= 0.322\ ;$$

$$q = 0.236 + 0.086 \times 0.091$$

$$= 0.236 + 0.008$$

$$= 0.244.$$

From Table 10, for m = 0.322 and q = 0.244 : $k = 0.447.$

(5) For entering Table 11, compute

$$\frac{1}{k} \times \frac{(2n-1)A_{sc}}{bd} = \frac{0.086}{0.447}$$

$$= 0.19,$$

$$\text{and } \frac{1}{k} \times \frac{d'}{d} = \frac{0.091}{0.447}$$

$$= 0.20.$$

From Table 11: $z = 0.30.$

(6) From Table 12, for z = 0.30 and k = 0.447 : $j = 0.87$ (actual).

* Even though k exceeds the transition value, f_{sc} may be less than F_{sc}, if f_s is also small by comparison with F_s. (see Example 18.)

$$\text{For } j = 0.87, \text{ determine } i = \frac{1}{1 - \frac{0.87}{-2.31}}$$

$$= 0.73.$$

$$\text{Then } f_s = \frac{1{,}000 \times (-45)}{0.87 \times 8.40 \times 0.73} \times (-2.31)$$

$$= 19{,}500 \text{ psi.;}$$

$$f_c = \frac{f_s}{n} \times \frac{k}{1-k}$$

$$= \frac{19{,}500}{12} \times \frac{0.447}{0.553}$$

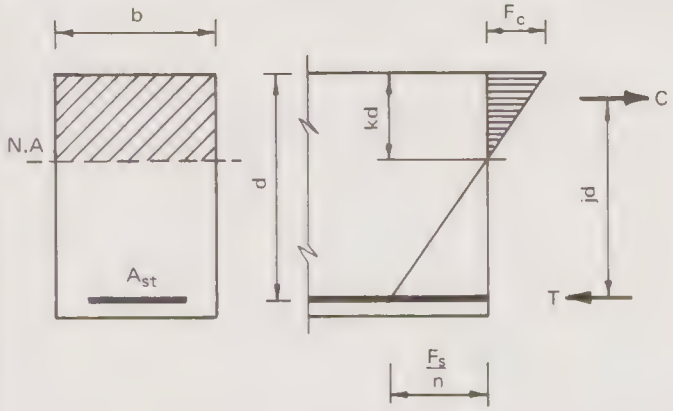
$$= 1{,}310 \text{ psi.;}$$

$$f_{sc} = \frac{2n'}{n} f_s \times \frac{k - \frac{d'}{d}}{1-k}$$

$$= \frac{15.8}{12} \times 19{,}500 \times \frac{0.447 - 0.091}{0.553}$$

$$= 16{,}500 \text{ psi.}$$

Table 1
Coefficients (k, j, K, p and a) for Rectangular Sections



$$k = \frac{1}{1 + \frac{F_s}{n F_c}}$$

$$j = 1 - \tfrac{1}{3}k$$

$$K = \tfrac{1}{2} F_c k j = \frac{1{,}000\, M_B}{bd^2}$$

$$p = \frac{A_{st}}{bd} = \frac{k F_c}{2 F_s}^{*}$$

$$\blacksquare = \frac{F_s j}{1{,}000}$$

$$\text{for use in } A_{st} = \frac{M}{ad}$$

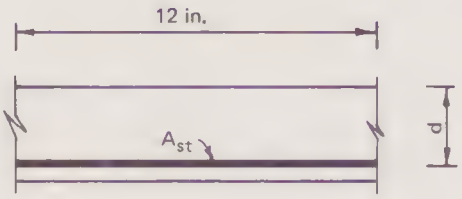
$$\text{or } A_{st} = \frac{Ne}{adi}$$

$\frac{F_c'}{F_c}$ n	F_s	k	j	K	p*	a
	15000	0.474	0.842	179	0.0142	12.63
2000	18000	0.429	0.857	165	0.0107	15.43
900	20000	0.403	0.866	156	0.0091	17.31
15	25000	0.351	0.883	139	0.0063	22.08
	30000	0.310	0.897	125	0.0047	26.90
	15000	0.494	0.835	231	0.0185	12.53
2500	18000	0.448	0.851	214	0.0140	15.31
1125	20000	0.422	0.859	204	0.0119	17.18
13	25000	0.369	0.877	182	0.0083	21.92
	30000	0.328	0.891	164	0.0061	26.72
	15000	0.519	0.827	289	0.0234	12.40
3000	18000	0.474	0.842	269	0.0178	15.16
1350	20000	0.448	0.851	257	0.0151	17.02
12	25000	0.393	0.869	230	0.0106	21.72
	30000	0.351	0.883	209	0.0079	26.49
	15000	0.536	0.821	346	0.0281	12.32
3500	18000	0.490	0.837	323	0.0215	15.06
1575	20000	0.464	0.845	308	0.0183	16.91
11	25000	0.409	0.864	278	0.0129	21.59
	30000	0.366	0.878	253	0.0096	26.34
	15000	0.545	0.818	401	0.0327	12.27
4000	18000	0.500	0.833	375	0.0250	15.00
1800	20000	0.474	0.842	359	0.0213	16.84
10	25000	0.419	0.860	324	0.0151	21.51
	30000	0.375	0.875	295	0.0112	26.25
	15000	0.574	0.809	522	0.0431	12.13
5000	18000	0.529	0.824	490	0.0331	14.82
2025	20000	0.503	0.832	471	0.0283	16.65
9	25000	0.448	0.851	428	0.0201	21.27
	30000	0.403	0.866	392	0.0151	25.97

Examples 1, 2, 3, 5, 6, 8, 9, 10 and 11 refer

* Balanced steel ratio — applicable to bending only.

Table 2
Resisting Moments (M_R) of Rectangular Sections 1 ft. Wide (Slabs)



$$M_R = \frac{12Kd^2}{1{,}000} \text{ (in. kips).}$$

(i) Enter table with known M or Ne (in. kips).

(ii) Select required effective depth d.

F'_c / F_c	n	F_s	Effective depth d (in.)																			
			2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7	7.5	8	9	10	11	12	14	16	18
		15000	8.59	13.4	19.3	26.3	34.4	43.5	53.7	65.0	77.3	90.8	105	121	137	174	215	260	309	421	550	696
2000		18000	7.92	12.4	17.8	24.3	31.7	40.1	49.5	59.9	71.3	83.7	97.0	111	127	160	198	240	285	388	507	642
		20000	7.49	11.7	16.9	22.9	30.0	37.9	46.8	56.6	67.4	79.1	91.7	105	120	152	187	227	270	367	479	607
900	15	25000	6.67	10.4	15.0	20.4	26.7	33.8	41.7	50.5	60.1	70.5	81.7	93.8	107	135	167	202	240	327	427	540
		30000	6.00	9.38	13.5	18.4	24.0	30.4	37.5	45.4	54.0	63.4	73.5	84.4	96.0	122	150	182	216	294	384	486
		15000	11.1	17.3	25.0	34.0	44.4	56.1	69.3	83.9	99.8	117	136	156	177	225	277	335	399	543	710	898
2500		18000	10.3	16.1	23.1	31.5	41.1	52.0	64.2	77.7	92.5	109	126	144	164	208	257	311	370	503	657	832
		20000	9.79	15.3	22.0	30.0	39.2	49.6	61.2	74.1	88.1	103	120	138	157	198	245	296	353	480	627	793
1125	13	25000	8.74	13.7	19.7	26.8	34.9	44.2	54.6	66.1	78.6	92.3	107	123	140	177	218	264	315	428	559	708
		30000	7.87	12.3	17.7	24.1	31.5	39.9	49.2	59.5	70.9	83.2	96.4	111	126	159	197	238	283	386	504	638
		15000	13.9	21.7	31.2	42.5	55.5	70.2	86.7	105	125	147	170	195	222	281	347	420	499	680	888	1124
3000		18000	12.9	20.2	29.1	39.5	51.7	65.4	80.7	97.7	116	136	158	182	207	261	323	391	465	633	826	1046
		20000	12.3	19.3	27.8	37.8	49.3	62.5	77.1	93.3	111	130	151	173	197	250	308	373	444	604	790	999
1350	12	25000	11.0	17.3	24.8	33.8	44.2	55.9	69.0	83.5	99.4	117	135	155	177	224	276	334	397	541	707	894
		30000	10.0	15.7	22.6	30.7	40.1	50.8	62.7	75.9	90.3	106	123	141	161	203	251	303	361	492	642	813
		15000	16.6	26.0	37.4	50.9	66.4	84.1	104	126	149	175	203	234	266	336	415	502	598	814	1063	1345
3500		18000	15.5	24.2	34.9	47.5	62.0	78.5	96.9	117	140	164	190	218	248	314	388	469	558	760	992	1256
		20000	14.8	23.1	33.3	45.3	59.1	74.8	92.4	112	133	156	181	208	237	299	370	447	532	724	946	1198
1575	11	25000	13.3	20.9	30.0	40.9	53.4	67.6	83.4	101	120	141	163	188	214	270	334	404	480	654	854	1081
		30000	12.1	19.0	27.3	37.2	48.6	61.5	75.9	91.8	109	128	149	171	194	246	304	367	437	595	777	984
		15000	19.3	30.1	43.3	59.0	77.0	97.4	120	146	173	203	236	271	308	390	481	582	693	943	1232	1559
4000		18000	18.0	28.1	40.5	55.1	72.0	91.1	113	136	162	190	221	253	288	365	450	545	648	882	1152	1458
		20000	17.2	26.9	38.8	52.8	68.9	87.2	108	130	155	182	211	242	276	349	431	521	620	844	1103	1396
1800	10	25000	15.6	24.3	35.0	47.6	62.2	78.7	97.2	118	140	164	191	219	249	315	389	470	560	762	995	1260
		30000	14.2	22.1	31.9	43.4	56.6	71.7	88.5	107	127	150	173	199	227	287	354	428	510	694	906	1147
		15000	25.1	39.2	56.4	76.7	100	127	157	189	226	265	307	352	401	507	626	758	902	1228	1604	2030
5000		18000	23.5	36.8	52.9	72.0	94.1	119	147	178	212	248	288	331	376	476	588	711	847	1152	1505	1905
		20000	22.6	35.3	50.9	69.2	90.4	114	141	171	203	239	277	318	362	458	565	684	814	1108	1447	1831
2025	9	25000	20.5	32.1	46.2	62.9	82.2	104	128	155	185	217	252	289	329	416	514	621	740	1007	1315	1664
		30000	18.8	29.4	42.3	57.6	75.3	95.3	118	142	169	199	231	265	301	381	470	569	677	922	1204	1524

Table 3
Round Bars* in Sections 1 ft. Wide (slabs)

Table 3a

Areas – A_{st} (sq. in.)											
Spacing (in.)	Bar Size No.										Spacing (in.)
	2	3	4	5	6	7	8	9	10	11	
2	.295	.663	1.18	1.84	2.65	3.61	4.71				2
2.5	.236	.530	.942	1.47	2.12	2.89	3.77	4.77	5.89		2.5
3	.196	.442	.785	1.23	1.77	2.41	3.14	3.98	4.91	5.94	3
3.5	.168	.379	.673	1.05	1.51	2.06	2.69	3.41	4.21	5.09	3.5
4	.147	.331	.589	.920	1.33	1.80	2.36	2.98	3.68	4.45	4
4.5	.131	.295	.524	.818	1.18	1.60	2.09	2.65	3.27	3.96	4.5
5	.118	.265	.471	.736	1.06	1.44	1.88	2.39	2.95	3.56	5
5.5	.107	.241	.428	.669	.964	1.31	1.71	2.17	2.68	3.24	5.5
6	.098	.221	.393	.614	.884	1.20	1.57	1.99	2.45	2.97	6
6.5	.091	.204	.362	.566	.816	1.11	1.45	1.84	2.27	2.74	6.5
7	.084	.189	.337	.526	.757	1.03	1.35	1.70	2.10	2.55	7
7.5	.079	.177	.314	.491	.707	.962	1.26	1.59	1.96	2.38	7.5
8	.074	.166	.295	.460	.663	.902	1.18	1.49	1.84	2.23	8
8.5	.069	.156	.277	.433	.624	.849	1.11	1.40	1.73	2.10	8.5
9	.065	.147	.262	.409	.589	.802	1.05	1.33	1.64	1.98	9
9.5	.062	.140	.248	.388	.558	.760	.992	1.26	1.55	1.88	9.5
10	.059	.133	.236	.368	.530	.722	.942	1.19	1.47	1.78	10
10.5	.056	.126	.224	.351	.505	.687	.898	1.14	1.40	1.70	10.5
11	.054	.120	.214	.335	.482	.656	.857	1.08	1.34	1.62	11
11.5	.051	.115	.205	.320	.461	.627	.820	1.04	1.28	1.55	11.5
12	.049	.110	.196	.307	.442	.601	.785	.994	1.23	1.48	12
13	.045	.102	.181	.283	.408	.555	.725	.918	1.13	1.37	13
14	.042	.095	.168	.263	.379	.515	.673	.852	1.05	1.27	14
15	.039	.088	.157	.245	.353	.481	.628	.795	.982	1.19	15
16	.037	.083	.147	.230	.331	.451	.589	.746	.920	1.11	16
17	.035	.078	.139	.217	.312	.424	.554	.702	.866	1.05	17
18	.033	.074	.131	.205	.295	.401	.524	.663	.818	.990	18
20	.029	.066	.118	.184	.265	.361	.471	.596	.736	.891	20

Example 1 refers

Table 3b

Perimeters – Σo (in.)											
Spacing (in.)	Bar Size No.										Spacing (in.)
	2	3	4	5	6	7	8	9	10	11	
2	4.71	7.07	9.42	11.8	14.1	16.5	18.8				2
2.5	3.77	5.65	7.54	9.42	11.3	13.2	15.1	17.0	18.8		2.5
3	3.14	4.71	6.28	7.85	9.42	11.0	12.6	14.1	15.7	17.3	3
3.5	2.69	4.04	5.39	6.73	8.08	9.42	10.8	12.1	13.5	14.8	3.5
4	2.36	3.53	4.71	5.89	7.07	8.25	9.42	10.6	11.8	13.0	4
4.5	2.09	3.14	4.19	5.24	6.28	7.33	8.38	9.42	10.5	11.5	4.5
5	1.89	2.83	3.77	4.71	5.65	6.60	7.54	8.48	9.42	10.4	5
5.5	1.71	2.57	3.43	4.28	5.14	6.00	6.85	7.71	8.57	9.42	5.5
6	1.57	2.36	3.14	3.93	4.71	5.50	6.28	7.07	7.85	8.64	6
6.5	1.45	2.17	2.90	3.62	4.35	5.07	5.80	6.52	7.25	7.97	6.5
7	1.35	2.02	2.69	3.37	4.04	4.71	5.39	6.06	6.73	7.41	7
7.5	1.26	1.89	2.51	3.14	3.77	4.40	5.03	5.65	6.28	6.91	7.5
8	1.18	1.77	2.36	2.95	3.53	4.12	4.71	5.30	5.89	6.48	8
8.5	1.11	1.66	2.22	2.77	3.33	3.88	4.44	4.99	5.54	6.10	8.5
9	1.05	1.57	2.09	2.62	3.14	3.67	4.19	4.71	5.24	5.76	9
9.5	.99	1.49	1.98	2.48	2.98	3.47	3.97	4.46	4.96	5.46	9.5
10	.94	1.41	1.89	2.36	2.83	3.30	3.77	4.24	4.71	5.18	10
10.5	.90	1.35	1.80	2.24	2.69	3.14	3.59	4.04	4.49	4.94	10.5
11	.86	1.29	1.71	2.14	2.57	3.00	3.43	3.86	4.28	4.71	11
11.5	.82	1.23	1.64	2.05	2.46	2.87	3.28	3.69	4.10	4.51	11.5
12	.79	1.18	1.57	1.96	2.36	2.75	3.14	3.53	3.93	4.32	12
13	.73	1.09	1.45	1.81	2.17	2.54	2.90	3.26	3.62	3.99	13
14	.67	1.01	1.35	1.68	2.02	2.36	2.69	3.03	3.37	3.70	14
15	.63	.94	1.26	1.57	1.89	2.20	2.51	2.83	3.14	3.46	15
16	.59	.88	1.18	1.47	1.77	2.06	2.36	2.65	2.95	3.24	16
17	.55	.83	1.11	1.39	1.66	1.94	2.22	2.49	2.77	3.05	17
18	.52	.79	1.05	1.31	1.57	1.83	2.09	2.36	2.62	2.88	18
20	.47	.71	.94	1.18	1.41	1.65	1.89	2.12	2.36	2.59	20

Example 1 refers

*Round bars refer to both plain and deformed round sections.

Table 3c

Minimum Reinforcement in Slabs

Minimum percent area of reinforcement (p) based upon total slab depth D :

$p\% = \frac{100\ A_s}{12\ D}$ (A_s in sq. in. per ft. width of slab).

Maximum pitch of main reinforcement = 3d (d = effective depth).
Maximum pitch of secondary reinforcement = 5d.

Area of secondary reinforcement not less than 20 percent of area of main reinforcement, per unit width of slab.

Procedure: Enter Table with known values of D, bar size and p%. Select maximum reinforcement pitch as the lesser of values given in Parts (i) and (ii) of the Table.

(i) Maximum Pitch Based on Effective Depth

D (in.)	3	3.5	4	4.5	5	5.5	6	6.5	7	7.5	8	8.5	9	9.5	10
Assumed d (in.)	2	2.5	2.9	3.4	3.9	4.4	4.9	5.4	5.7	6.2	6.5	7.0	7.5	8.0	8.5
Max. pitch 3d (main reinf.)	6	7.5	8.5	10	11.5	13	14	16	17	18	19	21	22	24	25
Max. pitch 5d (sec. reinf.)	10	12	14	17	19	22	24	27	28	31	32	35	37	40	42

(ii) Maximum Pitch Based on Minimum Area of Reinforcement

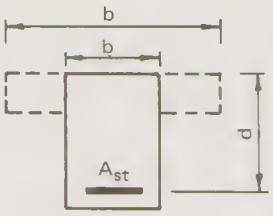
p%	D (in.)		3	3.5	4	4.5	5	5.5	6	6.5	7	7.5	8	8.5	9	9.5	10
0.15	A_s sq. in.		.054	.063	.072	.081	.090	.099	.108	.117	.126	.135	.144	.153	.162	.171	.180
	Pitch (in.)	3				16	14	13	12	11	10.5	9.5	9	8.5	8	7.5	7
	for bar	4							21	20	18	17	16	15	14	13	13
	size no.	5										27	25	24	22	21	20
0.18	A_s sq. in.		.065	.076	.086	.097	.108	.119	.130	.140	.151	.162	.173	.184	.194	.205	.216
	Pitch (in.)	3				13	12	11	10	9	8.5	8	7.5	7	6.5	6	6
	for bar	4						19	18	16	15	14	13	12	12	11.5	11
	size no.	5								26	24	22	21	20	19	18	17
0.20	A_s sq. in.		.072	.084	.096	.108	.120	.132	.144	.156	.168	.180	.192	.204	.216	.228	.240
	Pitch (in.)	3			13	12	11	10	9	8.5	7.5	7	6.5	6.5	6	5.5	5.5
	for bar	4					19	17	16	15	14	13	12	11.5	10.5	10	9.5
	size no.	5								23	21	20	19	18	17	16	15
0.22	A_s sq. in.		.079	.092	.106	.119	.132	.145	.158	.172	.185	.198	.211	.224	.238	.251	.264
	Pitch (in.)	3			12	11	10	9	8	8	7	6.5	6	5.5	5.5	5	5
	for bar	4						16	14	13	12	11.5	11	10.5	9.5	9	8.5
	size no.	5							23	21	19	18	17	16	15	14	13
0.25	A_s sq. in.		.090	.105	.120	.135	.150	.165	.180	.195	.210	.225	.240	.255	.270	.285	.300
	Pitch (in.)	3		12	11	9.5	8.5	8	7	6.5	6	5.5	5.5	5	4.5	4.5	4
	for bar	4				17	15	14	13	12	11	10.5	9.5	9	8.5	8	7.5
	size no.	5						22	20	18	17	16	15	14	13	12	12

Example 1 refers

Notes (i) Values of pitch based upon minimum A_s given in Part (ii) of the Table, are for reinforcement placed in a single layer.

(ii) Tabulated values of d are based upon maximum likely bar size for main reinforcement, and cover equal to the greater of : ¾ in., or bar diameter.

Table 4
Coefficients (F) for Resisting Moments of Rectangular and T–Sections



Values of $F = \frac{bd^2}{1{,}000} = \frac{M_R}{K}$.

(i) Enter table with required value of $F = \frac{M}{K}$ or $\frac{Ne}{K}$ (M, Ne in in. kips) and select b, d (in.)

(ii) Enter table with known b & d; compute $M_R = KF$ (K from below or Table 1).

d (in.)	Width of compressive area : b (in.)																					
	4	5	6	7	8	9	10	11	12	13	14	15	16	18	20	21	24	27	30	36	42	48
5	.100	125	150	175	.200	225	250	.275	300	325	350	375	.400	450	500	525	600	675	750	900	1 05	1 20
5.5	.121	151	181	212	242	.272	302	333	363	393	423	454	484	545	605	635	726	817	908	1 09	1 27	1 45
6	.144	.180	.216	.252	288	.324	360	396	.432	468	504	540	.576	648	720	756	864	972	1 08	1 30	1 51	1 73
6.5	.169	.211	253	296	.338	.380	.423	465	507	549	591	.634	676	761	845	887	1 01	1 14	1 27	1 52	1 77	2 03
7	.196	245	294	343	.392	.441	490	539	588	637	686	735	784	882	980	1 03	1 18	1 32	1 47	1 76	2 06	2 35
7.5	.225	281	338	394	450	.506	563	619	675	731	788	844	900	1 01	1 13	1 18	1 35	1 52	1 69	2 03	2 36	2 70
8	256	320	384	448	512	576	640	704	768	832	896	960	1 02	1 15	1 28	1 34	1 54	1 73	1 92	2 30	2 69	3 07
8.5	.289	361	433	506	578	.650	723	795	867	939	1 01	1 08	1 16	1 30	1 45	1 52	1 73	1 95	2 17	2 60	3 04	3 47
9	324	405	486	.567	648	729	810	891	972	1 05	1 13	1 22	1 30	1 46	1 62	1 70	1 94	2 19	2 43	2 92	3 40	3 89
9.5	361	451	542	632	722	812	902	993	1 08	1 17	1 26	1 35	1 44	1 63	1 81	1 90	2 17	2 44	2 71	3 25	3 79	4 33
10	400	500	600	700	800	900	1 00	1 10	1 20	1 30	1 40	1 50	1 60	1 80	2 00	2 10	2 40	2 70	3 00	3 60	4 20	4 80
10.5	441	551	662	772	882	992	1 10	1 21	1 32	1 43	1 54	1 65	1 76	1 98	2 21	2 32	2 65	2 98	3 31	3 97	4 63	5 29
11	484	605	726	847	968	1 09	1 21	1 33	1 45	1 57	1 69	1 82	1 94	2.18	2 42	2 54	2 90	3.27	3 63	4 36	5 08	5.81
11.5	529	661	793	926	1 06	1 19	1 32	1 46	1 59	1 72	1 85	1 98	2 12	2 38	2 65	2 78	3 17	3.57	3 97	4 76	5 55	6.35
12	576	720	864	1 01	1 15	1 30	1 44	1 58	1 73	1 87	2 02	2 16	2 30	2 59	2 88	3 02	3 46	3.89	4 32	5 18	6 05	6.91
13	676	845	1 01	1 18	1 35	1 52	1 69	1 86	2 03	2 20	2 37	2 54	2 70	3.04	3 38	3 55	4 06	4.56	5 07	6 08	7 10	8.11
14	784	980	1 18	1 37	1 57	1 76	1 96	2 16	2 35	2 55	2 74	2 94	3 14	3 52	3 92	4 12	4 70	5.29	5 88	7 06	8 23	9.41
15	900	1 13	1 35	1 58	1 80	2 03	2 25	2 48	2 70	2 93	3 15	3 38	3 60	4 05	4 50	4 73	5 40	6.08	6 75	8 10	9 45	10 8
16	1.02	1 28	1 54	1 79	2 05	2 30	2 56	2 82	3 07	3 33	3 58	3 84	4 10	4 61	5 12	5 38	6 14	6.91	7 68	9 22	10 8	12 3
17	1 16	1 45	1 73	2 02	2 31	2 60	2 89	3 18	3 47	3 76	4 05	4 34	4 62	5 20	5 78	6 07	6.94	7.80	8 67	10 4	12 1	13 9
18		1 62	1 94	2 27	2 59	2 92	3 24	3 56	3 89	4 21	4 54	4 86	5 18	5 83	6 48	6 80	7 78	8.75	9 72	11 7	13 6	15 6
19		1 81	2 17	2 53	2 89	3 25	3 61	3 97	4 33	4 69	5 05	5 42	5 78	6 50	7 22	7 58	8 66	9.75	10 8	13.0	15 2	17 3
20		2 00	2 40	2 80	3 20	3 60	4 00	4 40	4 80	5 20	5 60	6 00	6 40	7 20	8 00	8 40	9 60	10.8	12 0	14.4	16 8	19 2
21		2 21	2 65	3 09	3 53	3 97	4 41	4 85	5 29	5 73	6 17	6 62	7 06	7 94	8 82	9 26	10 6	11.9	13 2	15.9	18 5	21 2
22			2 90	3 39	3 87	4 36	4 84	5 32	5 81	6 29	6 78	7 26	7 74	8 71	9 68	10 2	11 6	13.1	14 5	17.4	20 3	23 2
23			3 17	3 70	4 23	4 76	5 29	5 82	6 35	6 88	7 41	7 94	8 46	9 52	10 6	11 1	12 7	14.3	15 9	19.0	22 2	25 4
24			3 46	4 03	4 61	5 18	5 76	6 34	6 91	7 49	8 06	8 64	9 22	10 4	11 5	12 1	13 8	15.6	17 3	20.7	24 2	27 6
25			3 75	4 38	5 00	5 63	6 25	6 88	7 50	8 13	8 75	9 38	10 0	11 3	12 5	13 1	15 0	16.9	18 8	22.5	26 3	30 0
26				4 73	5 41	6 08	6 76	7 44	8 11	8 79	9 46	10 1	10 8	12 2	13 5	14 2	16 2	18.3	20 3	24.3	28 4	32 5
27				5 10	5 83	6 56	7 29	8 02	8 75	9 48	10 2	10 9	11 7	13 1	14 6	15 3	17 5	19.7	21 9	26.2	30 6	35 0
28				5 49	6 27	7 06	7 84	8 62	9 41	10 2	11 0	11 8	12 5	14 1	15 7	16 5	18 8	21.2	23 5	28.2	32 9	37 6
29				5 89	6 73	7 57	8 41	9 25	10 1	10 9	11 8	12 6	13 5	15 1	16 8	17 7	20 2	22.7	25 2	30.3	35 3	40 4
30					7 20	8 10	9 00	9 90	10 8	11 7	12 6	13 5	14 4	16 2	18 0	18 9	21 6	24 3	27.0	32.4	37 8	43 2
31					7 69	8 65	9 61	10 6	11 5	12 5	13 5	14 4	15 4	17 3	19 2	20.2	23 1	25 9	28.8	34.6	40 4	46 1
32					8 19	9 22	10 2	11 3	12 3	13 3	14 3	15 4	16 4	18 4	20 5	21.5	24 6	27 6	30.7	36.9	43 0	49 2
33					8 77	9 80	10 9	12 0	13 1	14 2	15 2	16 3	17 4	19 6	21 8	22.9	26 1	29 4	32.7	39.2	45 7	52 3
34						10 4	11 6	12 7	13 9	15 0	16 2	17 3	18 5	20 8	23 1	24.3	27 7	31 2	34.7	41 6	48 6	55 5
36						11 7	13 0	14 3	15 6	16 8	18 1	19 4	20 7	23 3	25 9	27.2	31 1	35 0	38.9	46 7	54 4	62 2
38						13 0	14 4	15 9	17 3	18 6	20 2	21 7	23 1	26 0	28 9	30.3	34 7	39 0	43.3	52 0	60 6	69 3
40						14 4	16 0	17 6	19 2	20 8	22 4	24 0	25 6	28 8	32 0	33.6	38 4	43 2	48.0	57 6	67 2	76 8
42						15 9	17 6	19 4	21 2	22 9	24 7	26 5	28 2	31 8	35 3	37.0	42.3	47 6	52.9	63 5	74 1	84 7
44						17 4	19 4	21 5	23 2	25 2	27 1	29 0	31 0	34 8	38 7	40 7	46.5	52 3	58 1	69 7	81 3	92 9
46						19 0	21 2	23 3	25 4	27 5	29 6	31 7	33 9	38 1	42 3	44 4	50.8	57 1	63 5	76 2	88 9	102
48						20 7	23 0	25 0	27 6	30 0	32 3	34 6	36 9	41 5	46 1	48 4	55.3	62 2	69 1	82 9	96 8	111
50							25.0	27.5	30.0	32.5	35.0	37.5	40.0	45.0	50.0	52.5	60.0	67.5	75.0	90.0	105	120
52							27.0	29.7	32.4	35.2	37.9	40.6	43.3	48.7	54.1	56.8	64.9	73.0	81.1	97.3	114	130
54							29.2	32.1	35.0	37.9	40.8	43.7	46.7	52.5	58.3	61.2	70.0	78.7	87.5	105	122	140
56							31.4	34.5	37.6	40.8	43.9	47.0	50.2	56.4	62.7	65.9	75.3	84.7	94.1	113	132	151
60							36.1	39.6	43.2	46.8	50.4	54.0	57.6	64.8	72.0	75.6	86.4	97.2	108	130	151	173
64							41.6	45.1	49.2	53.2	57.3	61.4	65.5	73.7	81.9	86.0	98.3	111	123	147	172	197
68							46.2	50.9	55.5	60.1	64.7	69.4	74.0	83.2	92.5	97.1	111	125	139	166	194	222
72							51.8	57.0	62.2	67.4	72.6	77.8	82.9	93.3	104	109	124	140	156	187	218	249

K – values*

F_s	F_c					
(ksi.)	2000	2500	3000	3500	4000	5000
15	179	231	289	340	401	522
18	165	214	269	323	375	490
20	156	204	257	306	359	471
25	139	182	230	278	324	428
30	125	164	209	253	295	392

*Values of K correspond to $F_c = 0.45\ F'_c$.

Examples 2, 3, 4, 5, 6, 7, 8 and 9 refer

Table 5 Properties of Round Bars

Table 5a

Nominal Size and Weight of Bars											
Bar Size No.	2	3	4	5	6	7	8	9	10	11	Bar Size No.
Diameter (in.)	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	Diameter (in.)
Weight (lb. per ft.)	.167	.376	.668	1.043	1.502	2.044	2.670	3.380	4.172	5.049	Weight (lb. per ft.)

Table 5b

Areas — A_s (sq. in.)											
No. of Bars	Bar Size No.										No. of Bars
	2	3	4	5	6	7	8	9	10	11	
1	.049	.110	.196	.307	.442	.601	.785	.994	1.23	1.48	1
2	.098	.221	.393	.614	.884	1.20	1.57	1.99	2.45	2.97	2
3	.147	.331	.589	.920	1.33	1.80	2.36	2.98	3.68	4.45	3
4	.196	.442	.785	1.23	1.77	2.41	3.14	3.98	4.91	5.94	4
5	.245	.552	.982	1.53	2.21	3.01	3.93	4.97	6.14	7.42	5
6	.295	.663	1.18	1.84	2.65	3.61	4.71	5.96	7.36	8.91	6
7	.344	.773	1.37	2.15	3.09	4.21	5.50	6.96	8.59	10.4	7
8	.393	.884	1.57	2.45	3.53	4.81	6.28	7.95	9.82	11.9	8
9	.442	.994	1.77	2.76	3.98	5.41	7.07	8.95	11.0	13.4	9
10	.491	1.10	1.96	3.07	4.42	6.01	7.85	9.94	12.3	14.8	10
11	.540	1.21	2.16	3.37	4.86	6.61	8.64	10.9	13.5	16.3	11
12	.589	1.33	2.36	3.68	5.30	7.22	9.42	11.9	14.7	17.8	12
13	.638	1.44	2.55	3.99	5.74	7.82	10.2	12.9	17.2	19.3	13
14	.687	1.55	2.75	4.30	6.19	8.42	11.0	13.9	19.2	20.8	14
15	.736	1.66	2.95	4.60	6.63	9.02	11.8	14.9	18.4	22.3	15
16	.785	1.77	3.14	4.91	7.07	9.62	12.6	15.9	19.6	23.8	16
17	.834	1.88	3.34	5.22	7.51	10.2	13.4	16.9	20.9	25.2	17
18	.884	1.99	3.53	5.52	7.95	10.8	14.1	17.9	22.1	26.7	18
19	.933	2.10	3.73	5.83	8.39	11.4	14.9	18.9	23.3	28.2	19
20	.982	2.21	3.93	6.14	8.84	12.0	15.7	19.9	24.5	29.7	20
21	1.03	2.32	4.12	6.44	9.28	12.6	16.5	20.9	25.8	31.2	21
22	1.08	2.43	4.32	6.75	9.72	13.2	17.3	21.9	27.0	32.7	22
23	1.13	2.54	4.52	7.06	10.2	13.8	18.1	22.9	28.2	34.2	23
24	1.18	2.65	4.71	7.36	10.6	14.4	18.8	23.9	29.5	35.6	24

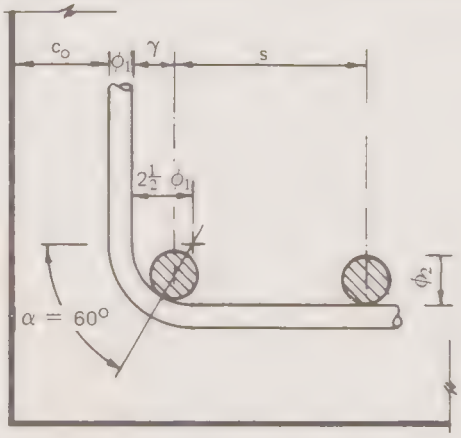
Example 2 refers

Table 5c

Perimeters — Σo (in.)											
No. of Bars	Bar Size No.										No. of Bars
	2	3	4	5	6	7	8	9	10	11	
1	0.79	1.18	1.57	1.96	2.36	2.75	3.14	3.53	3.93	4.32	1
2	1.57	2.36	3.14	3.93	4.71	5.50	6.28	7.07	7.85	8.64	2
3	2.36	3.53	4.71	5.89	7.07	8.25	9.42	10.6	11.8	13.0	3
4	3.14	4.71	6.28	7.85	9.42	11.0	12.6	14.1	15.7	17.3	4
5	3.93	5.89	7.85	9.82	11.8	13.7	15.7	17.7	19.6	21.6	5
6	4.71	7.07	9.42	11.8	14.1	16.5	18.8	21.2	23.6	25.9	6
7	5.50	8.25	11.0	13.7	16.5	19.2	22.0	24.7	27.5	30.2	7
8	6.28	9.42	12.6	15.7	18.8	22.0	25.1	28.3	31.4	34.6	8
9	7.07	10.6	14.1	17.7	21.2	24.7	28.3	31.8	35.3	38.9	9
10	7.85	11.8	15.7	19.6	23.6	27.5	31.4	35.3	39.3	43.2	10
11	8.64	13.0	17.3	21.6	25.9	30.2	34.6	38.9	43.2	47.5	11
12	9.42	14.1	18.8	23.6	28.3	33.0	37.7	42.4	47.1	51.8	12
13	10.2	15.3	20.4	25.5	30.6	35.7	40.8	45.9	51.1	56.2	13
14	11.0	16.5	22.0	27.5	33.0	38.5	44.0	49.5	55.0	60.5	14
15	11.8	17.7	23.6	29.5	35.3	41.2	47.1	53.0	58.9	64.8	15
16	12.6	18.8	25.1	31.4	37.7	44.0	50.3	56.5	62.8	69.1	16
17	13.4	20.0	26.7	33.4	40.1	46.7	53.4	60.1	66.8	73.4	17
18	14.1	21.2	28.3	35.3	42.4	49.5	56.5	63.6	70.7	77.8	18
19	14.9	22.4	29.8	37.3	44.8	52.2	59.7	67.2	74.6	82.1	19
20	15.7	23.6	31.4	39.3	47.1	55.0	62.8	70.7	78.5	86.4	20
21	16.5	24.7	33.0	41.2	49.5	57.7	66.0	74.2	82.5	90.7	21
22	17.3	25.9	34.6	43.2	51.8	60.5	69.1	77.8	86.4	95.0	22
23	18.1	27.1	36.1	45.2	54.2	63.2	72.3	81.3	90.3	99.4	23
24	18.8	28.3	37.7	47.1	56.5	66.0	75.4	84.8	94.2	104	24

Example 2 refers

Table 6
Minimum Web Widths (b_w) for a Given Number of Bars in a Row



Web Width = b_w (in.)
= $2(c_o + \phi_1 + \gamma) + (x - 1)s$,

where x = number of bars in row
c_o = clear cover
$s - \phi_2$ = bar spacing
ϕ_1 = dia. of stirrup (in.)
ϕ_2 = dia. of main bar (in.)
γ = $0.25\ (5\phi_1 + \phi_2)$.

Table conditions: (i) beams in sheltered locations
(ii) $\frac{3}{4}$ in. max. aggregate size
(iii) $\frac{3}{8}$ in. dia. stirrups (ϕ_1)
(iv) b_w rounded to $\frac{1}{2}$ in. above

Number of bars	Main Reinforcement Size No. (ϕ_2)							
	4	5	6	7	8	9	10	11
1*	5.0	5.0	5.0	5.0	5.0	5.0	5.5	5.5
2*	6.5	6.5	6.5	7.0	7.0	7.5	8.0	8.5
3**	8.0	8.0	8.5	8.5	9.0	9.5	10.0	11.0
4	9.0	9.5	10.0	10.5	11.0	11.5	12.5	13.5
5	10.5	11.0	12.0	12.5	13.0	13.5	15.0	16.5
6	12.5	13.0	13.5	14.5	15.0	16.0	17.5	19.0
7	14.0	14.5	15.5	16.5	17.0	18.0	20.0	22.0
8	15.5	16.5	17.5	18.5	19.5	20.5	22.5	24.5
9	17.0	18.0	19.5	20.5	21.5	22.5	25.0	27.5
10	19.0	20.0	21.0	22.5	23.5	25.0	27.5	30.0

Example 2 refers

* Effective depth value of corner bars = full value ($\alpha = 90^o$).
**Effective depth value of corner bars = transitional value ($60^o < \alpha < 90^o$).

Notes: (i) For ¼ in. dia. stirrups decrease table values for b_w by ½ in.
(ii) For ½ in. dia. stirrups increase table values for b_w by ½ in.
(iii) For beams exposed to weather, fresh water or ground water, increase table values by 1 in.
(iv) For beams in contact with ground, increase table values by 3 in.

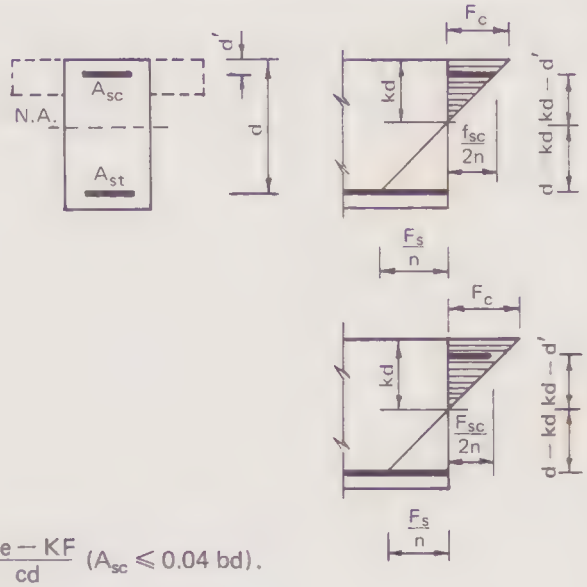
Table 7
Coefficients (c) for Compressive Reinforcement for Rectangular and T–Sections

For values in light print
$k \leqslant$ transition value and

$$c = \frac{F_s\,(2n-1)(1-\frac{d'}{d})(k-\frac{d'}{d})}{1{,}000\,n(1-k)}$$

For values in bold print
$k >$ transition value and

$$c = \frac{F_{sc}\,[1-\frac{F_c}{F_{sc}}(1-\frac{d'}{kd})](1-\frac{d'}{d})}{1{,}000}$$



Procedure:
Enter table with known values of $\frac{d'}{d}$, F_s & F_c;
select value of c and compute $A_{sc} = \frac{M-KF}{cd}$ or $\frac{Ne-KF}{cd}$ $(A_{sc} \leqslant 0.04\ bd)$.

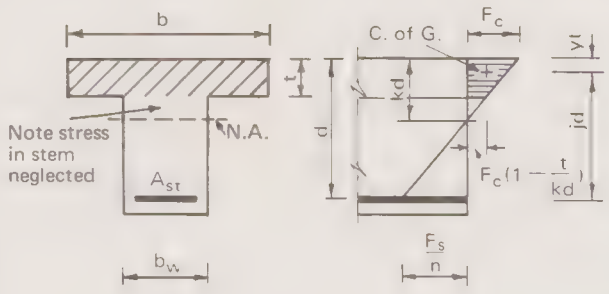
Transition Values for k ($f_{sc} = F_{sc}$)

F_s	15000	18000	20000	25000	30000
F_{sc}	15000	15000	16500	20000	23000
K	$0.333 + 0.667\frac{d'}{d}$	$0.294 + 0.706\frac{d'}{d}$	$0.292 + 0.708\frac{d'}{d}$	$0.286 + 0.714\frac{d'}{d}$	$0.277 + 0.723\frac{d'}{d}$

$\frac{F'_c}{F_c}$ n	F_c	F_{sc}	$\frac{d'}{d}$ .02	.04	.06	.08	.10	.12	.14	.16	.18	.20
2000	15000	15000	13.9	13.6	13.4	13.1	12.9	12.6	12.4	12.1	11.8	11.6
	18000	15000	13.9	13.6	13.4	13.1	12.9	12.6	12.4	12.1	11.9	11.2
900	20000	16500	15.3	15.1	14.8	14.5	14.2	14.0	13.7	13.2	11.8	10.5
	25000	20000	18.8	18.4	18.1	17.8	16.8	15.1	13.5	11.9	10.4	9.0
15	30000	23000	21.7	21.3	19.8	17.8	15.9	14.1	12.3	10.6	9.0	7.4
2500	15000	15000	13.6	13.4	13.2	12.9	12.7	12.5	12.2	12.0	11.7	11.5
	18000	15000	13.6	13.4	13.2	12.9	12.7	12.5	12.2	12.0	11.7	11.5
1125	20000	16500	15.1	14.9	14.6	14.3	14.1	13.8	13.5	13.3	13.0	11.8
	25000	20000	18.6	18.2	17.9	17.6	17.3	16.7	15.0	13.4	11.8	10.3
13	30000	23000	21.5	21.1	20.8	19.6	17.6	15.7	13.9	12.1	10.4	8.8
3000	15000	15000	13.4	13.2	13.0	12.7	12.5	12.3	12.1	11.8	11.6	11.3
	18000	15000	13.4	13.2	13.0	12.8	12.5	12.3	12.1	11.8	11.6	11.4
1350	20000	16500	14.9	14.7	14.4	14.2	13.9	13.7	13.4	13.1	12.9	12.6
	25000	20000	18.3	18.0	17.7	17.4	17.1	16.8	16.5	15.5	13.8	12.2
12	30000	23000	21.3	20.9	20.6	20.2	19.8	18.0	16.1	14.2	12.4	10.7
3500	15000	15000	13.2	13.0	12.8	12.6	12.3	12.1	11.9	11.7	11.4	11.2
	18000	15000	13.2	13.0	12.8	12.6	12.4	12.2	11.9	11.7	11.5	11.3
1575	20000	16500	14.7	14.5	14.2	14.0	13.7	13.5	13.2	13.0	12.7	12.5
	25000	20000	18.1	17.8	17.5	17.2	16.9	16.6	16.3	16.0	15.2	13.5
11	30000	23000	21.0	20.7	20.4	20.0	19.7	19.3	17.6	15.6	13.8	12.0
4000	15000	15000	13.0	12.8	12.6	12.4	12.2	12.0	11.7	11.5	11.3	11.1
	18000	15000	13.0	12.8	12.6	12.4	12.2	12.0	11.8	11.6	11.4	11.1
1800	20000	16500	14.5	14.3	14.0	13.8	13.6	13.3	13.1	12.9	12.6	12.4
	25000	20000	17.9	17.6	17.4	17.1	16.8	16.5	16.2	15.9	15.6	14.3
10	30000	23000	20.9	20.5	20.2	19.9	19.5	19.2	18.4	16.5	14.6	12.8
5000	15000	15000	12.6	12.4	12.2	12.0	11.8	11.6	11.4	11.2	11.0	10.8
	18000	15000	12.6	12.4	12.2	12.0	11.9	11.7	11.5	11.3	11.1	10.9
2250	20000	16500	14.1	13.9	13.6	13.4	13.2	13.0	12.8	12.6	12.3	12.1
	25000	20000	17.5	17.2	17.0	16.7	16.4	16.2	15.9	15.6	15.3	15.0
9	30000	23000	20.4	20.1	19.8	19.5	19.2	18.8	18.5	18.2	17.4	15.4

Examples 3, 5, 6, 8 and 9 refer

Table 8
Coefficients (K, j and y) for T–Sections with Tensile Reinforcement Only



For $t \leqslant kd$:

$$j = 1 - \frac{k - \frac{2t}{3d}}{2k - \frac{t}{d}} \times \frac{t}{d}$$

$$= 1 - y\frac{t}{d}$$

$$K = \frac{F_c}{2} \times \frac{t}{d} \times (2 - \frac{t}{d} - \frac{t}{kd} + \frac{2t^2}{3kd^2})$$

For $t > kd$:

$$j = 1 - \frac{k}{3}$$

$$K = \frac{F_c}{2} \times kj$$

Procedure:

Enter table with known values of $\frac{t}{d}$, F_s & F_c and select value of K or j for use in:

$$A_{st} = \frac{1{,}000\,M}{F_s jdi}, \quad A_{sc} = \frac{M - KF}{cd} \text{ or } A_{sc} = \frac{Ne - KF}{cd}.$$

$\frac{t}{kd}$	0.15	0.20	0.25	0.30	0.35	0.40	0.45	0.50	0.55	0.60	0.65	0.70	0.75	0.80	0.85	0.90	0.95	1.00
y	0.49	0.48	0.48	0.47	0.46	0.46	0.45	0.44	0.44	0.43	0.42	0.41	0.40	0.39	0.38	0.36	0.35	0.33

F_c' / F_c / n	F_s		$\frac{t}{d}$ .06	.08	.10	.12	.14	.16	.18	.20	.24	.28	.32	.36	.40
2000	15000	K	49	63	77	89	100	111	120	129	144	156	165	172	177
		j	.97	.96	.95	.94	.93	.93	.92	.91	.89	.88	.87	.86	.85
	18000	K	49	63	76	88	99	108	118	126	139	150	157	162	165
		j	.97	.96	.95	.94	.93	.93	.92	.91	.90	.88	.87	.86	.86
900	20000	K	49	62	75	87	97	107	116	123	136	146	152	156	157
		j	.97	.96	.95	.94	.93	.93	.92	.91	.90	.88	.88	.87	.87
	25000	K	48	61	74	85	94	103	111	118	128	135	139	139	139
15		j	.97	.96	.95	.94	.94	.93	.92	.91	.90	.89	.88	.88	.88
	30000	K	47	60	72	82	91	99	106	112	120	124	125	125	125
		j	.97	.96	95	.94	.94	.93	.92	.92	.91	.90	.90	.90	.90
2500	15000	K	62	80	96	112	126	140	152	163	183	198	211	220	226
		j	.97	.96	.95	.94	.93	.93	.92	.91	.89	.88	.87	.85	.85
	18000	K	61	79	95	110	124	137	148	159	177	191	201	208	213
		j	.97	.96	.95	.94	.93	.93	.92	.91	.89	.88	.87	.86	.85
1125	20000	K	61	78	94	109	123	135	146	156	173	186	195	201	204
		j	.97	.96	.95	.94	.93	.93	.92	.91	.90	.88	.87	.86	.86
	25000	K	60	77	93	107	119	131	141	150	164	174	180	182	182
13		j	.97	.96	.95	.94	.94	.93	.92	.91	.90	.89	.88	.88	.88
	30000	K	60	76	91	104	116	126	135	143	155	162	164	164	164
		j	.97	.96	.95	.94	.94	.93	.92	.91	.90	.89	.89	.89	.89

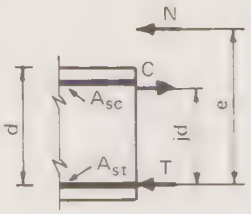
Examples 4, 5, 7, 14, 15, 16, 19, 20 and 21 refer

continued

Table 8 (ctd.)

F'_c / F_c / n	F_s		t/d .06	.08	.10	.12	.14	.16	.18	.20	.24	.28	.32	.36	.40
	15000	K	74	96	116	135	153	169	184	198	222	242	258	270	279
		j	.97	.96	.95	.94	.93	.92	.92	.91	.89	.88	.86	.85	.84
3000	18000	K	74	95	115	133	150	166	181	194	216	234	248	258	265
		j	.97	.96	.95	.94	.93	.93	.92	.91	.89	.88	.87	.86	.85
1350	20000	K	73	95	114	132	149	164	178	191	212	229	242	250	255
		j	.97	.96	.95	.94	.93	.93	.92	.91	.89	.88	.87	.86	.85
12	25000	K	73	93	112	130	145	159	172	183	202	216	225	229	230
		j	.97	.96	.95	.94	.94	.93	.92	.91	.90	.89	.88	.87	.87
	30000	K	72	92	110	127	142	155	166	176	192	202	208	209	209
		j	.97	.96	.95	.94	.94	.93	.92	.91	.90	.89	.88	.88	.88
	15000	K	87	112	136	158	179	198	216	233	262	286	305	320	332
		j	.97	.96	.95	.94	.93	.92	.92	.91	.89	.88	.86	.85	.84
3500	18000	K	86	111	135	156	177	195	212	228	255	277	294	307	315
		j	.97	.96	.95	.94	.93	.93	.92	.91	.89	.88	.87	.85	.85
1575	20000	K	86	111	134	155	175	193	210	225	251	271	287	298	305
		j	.97	.96	.95	.94	.93	.93	.92	.91	.89	.88	.87	.86	.85
11	25000	K	85	109	132	152	171	188	203	217	239	256	268	275	278
		j	.97	.96	.95	.94	.93	.93	.92	.91	.90	.88	.87	.87	.86
	30000	K	84	108	130	149	167	183	197	209	229	242	250	253	253
		j	.97	.96	.95	.94	.94	.93	.92	.91	.90	.89	.88	.88	.88
	15000	K	99	128	156	181	205	227	248	267	300	328	351	369	382
		j	.97	.96	.95	.94	.93	.92	.92	.91	.89	.88	.86	.85	.84
4000	18000	K	99	127	154	179	202	224	244	262	293	319	339	354	365
		j	.97	.96	.95	.94	.93	.93	.92	.91	.89	.88	.87	.85	.84
1800	20000	K	98	127	153	178	201	222	241	258	288	312	331	344	353
		j	.97	.96	.95	.94	.93	.93	.92	.91	.89	.88	.87	.86	.85
10	25000	K	97	125	151	175	196	216	234	250	276	296	311	320	324
		j	.97	.96	.95	.94	.93	.93	.92	.91	.90	.88	.87	.87	.86
	30000	K	96	124	149	171	192	210	226	241	264	280	291	295	295
		j	.97	.96	.95	.94	.94	.93	.92	.91	.90	.89	.88	.88	.88
	15000	K	124	161	195	228	258	286	313	337	380	417	447	471	490
		j	.97	.96	.95	.94	.93	.92	.92	.91	.89	.88	.86	.85	.84
5000	18000	K	124	160	194	226	255	283	308	331	372	406	433	455	470
		j	.97	.96	.95	.94	.93	.92	.92	.91	.89	.88	.86	.85	.84
2250	20000	K	123	159	193	224	253	280	305	327	367	399	425	444	458
		j	.97	.96	.95	.94	.93	.93	.92	.91	.89	.88	.86	.85	.84
9	25000	K	122	158	190	221	248	274	297	318	354	382	403	417	425
		j	.97	.96	.95	.94	.93	.93	.92	.91	.89	.88	.87	.86	.85
	30000	K	121	156	188	217	243	267	289	308	340	364	380	389	392
		j	.97	.96	.95	.94	.93	.93	.92	.91	.90	.88	.88	.87	.87

Table 9
Coefficients (i) for Flexural Members Subject to Combined Bending and Axial Loads



Values of $i = \dfrac{1}{1 - \dfrac{jd}{e}}$ (for $e > jd$)

Enter table with known values of $\frac{e}{d}$ & j; select i for use in:

$A_{st} = \dfrac{Ne}{adi}$, or m and q in Table 10.

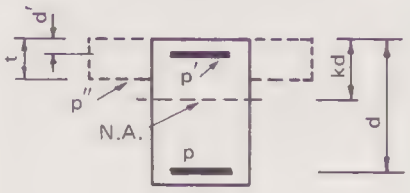
For guide in estimating j in investigation follow the zig-zag line.*

e/d	j																			
	.75	.76	.77	.78	.79	.80	.81	.82	.83	.84	.85	.86	.87	.88	.89	.90	.91	.92	.93	.94	.95
1.15	2.87	2.95	3.03	3.11	3.20	3.29	3.38	3.48	3.59	3.71	3.83	3.96	4.11	4.26	4.43	4.60	4.79	5.00	5.23	5.48	5.75
1.16	2.83	2.90	2.98	3.05	3.13	3.22	3.31	3.41	3.52	3.62	3.74	3.87	4.00	4.14	4.30	4.46	4.64	4.84	5.04	5.27	5.52
1.17	2.79	2.85	2.92	3.00	3.08	3.16	3.25	3.34	3.44	3.55	3.66	3.77	3.90	4.03	4.18	4.33	4.50	4.68	4.87	5.09	5.32
1.18	2.75	2.81	2.88	2.95	3.02	3.11	3.19	3.28	3.37	3.47	3.58	3.69	3.81	3.93	4.07	4.21	4.36	4.54	4.72	4.92	5.13
1.19	2.70	2.77	2.83	2.90	2.98	3.05	3.13	3.22	3.31	3.40	3.50	3.61	3.72	3.84	3.97	4.10	4.25	4.41	4.58	4.76	4.96
1.20	2.67	2.73	2.79	2.86	2.93	3.00	3.08	3.16	3.24	3.33	3.42	3.52	3.64	3.75	3.87	4.00	4.14	4.29	4.44	4.62	4.80
1.22	2.60	2.65	2.71	2.77	2.84	2.91	2.98	3.05	3.13	3.21	3.30	3.39	3.49	3.59	3.70	3.81	3.94	4.07	4.21	4.36	4.52
1.24	2.53	2.58	2.64	2.70	2.76	2.82	2.88	2.95	3.03	3.10	3.18	3.26	3.35	3.44	3.54	3.65	3.76	3.89	4.03	4.18	4.33
1.26	2.47	2.52	2.57	2.62	2.68	2.74	2.80	2.86	2.93	3.00	3.07	3.15	3.23	3.32	3.41	3.50	3.60	3.71	3.82	3.94	4.06
1.28	2.41	2.46	2.51	2.56	2.61	2.67	2.72	2.78	2.84	2.91	2.98	3.05	3.12	3.21	3.28	3.37	3.46	3.56	3.66	3.76	3.88
1.30	2.36	2.41	2.45	2.50	2.55	2.60	2.65	2.71	2.77	2.83	2.89	2.96	3.03	3.10	3.17	3.25	3.33	3.42	3.51	3.61	3.71
1.32	2.32	2.36	2.40	2.44	2.49	2.54	2.59	2.64	2.69	2.75	2.81	2.87	2.93	3.00	3.07	3.14	3.22	3.30	3.38	3.47	3.57
1.34	2.27	2.31	2.35	2.39	2.44	2.48	2.53	2.58	2.63	2.68	2.74	2.79	2.85	2.92	2.98	3.05	3.12	3.19	3.27	3.35	3.44
1.36	2.23	2.27	2.31	2.35	2.39	2.43	2.47	2.52	2.57	2.62	2.67	2.72	2.78	2.83	2.89	2.96	3.02	3.09	3.16	3.24	3.32
1.38	2.19	2.23	2.26	2.30	2.34	2.38	2.42	2.46	2.51	2.56	2.61	2.65	2.71	2.76	2.82	2.88	2.94	3.00	3.07	3.14	3.21
1.40	2.15	2.19	2.22	2.26	2.30	2.33	2.37	2.41	2.46	2.50	2.55	2.59	2.64	2.69	2.75	2.80	2.86	2.92	2.98	3.04	3.11
1.42	2.12	2.15	2.18	2.22	2.25	2.29	2.33	2.37	2.41	2.45	2.49	2.54	2.58	2.63	2.68	2.73	2.79	2.84	2.90	2.96	3.02
1.44	2.09	2.12	2.15	2.18	2.22	2.25	2.28	2.32	2.36	2.40	2.44	2.49	2.53	2.57	2.62	2.67	2.72	2.77	2.82	2.88	2.94
1.46	2.06	2.09	2.12	2.15	2.18	2.21	2.25	2.28	2.32	2.35	2.39	2.43	2.48	2.52	2.56	2.61	2.66	2.71	2.75	2.81	2.86
1.48	2.03	2.06	2.09	2.12	2.15	2.18	2.21	2.24	2.28	2.31	2.35	2.39	2.43	2.47	2.51	2.55	2.60	2.64	2.69	2.74	2.79
1.50	2.00	2.03	2.05	2.08	2.11	2.14	2.17	2.21	2.24	2.27	2.31	2.34	2.38	2.42	2.46	2.50	2.54	2.59	2.63	2.68	2.73
1.55	1.94	1.96	1.99	2.01	2.04	2.07	2.10	2.12	2.15	2.18	2.21	2.25	2.28	2.31	2.35	2.38	2.42	2.46	2.50	2.54	2.58
1.60	1.88	1.90	1.93	1.95	1.98	2.00	2.03	2.05	2.08	2.10	2.13	2.16	2.19	2.22	2.25	2.29	2.32	2.35	2.39	2.42	2.46
1.65	1.83	1.85	1.88	1.90	1.92	1.94	1.97	1.99	2.01	2.04	2.06	2.09	2.11	2.14	2.17	2.20	2.23	2.26	2.29	2.33	2.36
1.70	1.79	1.81	1.83	1.85	1.87	1.89	1.91	1.93	1.96	1.98	2.00	2.03	2.05	2.07	2.10	2.13	2.15	2.18	2.21	2.24	2.27
1.75	1.75	1.77	1.78	1.80	1.82	1.84	1.86	1.88	1.90	1.92	1.94	1.97	1.99	2.01	2.04	2.06	2.08	2.11	2.14	2.16	2.19
1.80	1.71	1.73	1.75	1.76	1.78	1.80	1.82	1.84	1.86	1.87	1.89	1.91	1.93	1.95	1.98	2.00	2.02	2.04	2.07	2.09	2.12
1.85	1.68	1.70	1.71	1.73	1.75	1.76	1.78	1.80	1.81	1.83	1.85	1.87	1.89	1.91	1.93	1.95	1.97	1.99	2.01	2.03	2.06
1.90	1.65	1.67	1.68	1.70	1.71	1.73	1.74	1.76	1.78	1.79	1.81	1.83	1.84	1.86	1.88	1.90	1.92	1.94	1.96	1.98	2.00
1.95	1.62	1.64	1.65	1.67	1.68	1.70	1.71	1.73	1.74	1.76	1.77	1.79	1.81	1.82	1.84	1.86	1.88	1.89	1.91	1.93	1.95
2.0	1.60	1.61	1.63	1.64	1.65	1.67	1.68	1.69	1.71	1.72	1.74	1.76	1.77	1.79	1.80	1.82	1.84	1.85	1.87	1.89	1.91
2.1	1.56	1.57	1.58	1.59	1.60	1.62	1.63	1.64	1.65	1.67	1.68	1.69	1.71	1.72	1.74	1.75	1.77	1.78	1.80	1.81	1.83
2.2	1.52	1.53	1.54	1.55	1.56	1.57	1.58	1.59	1.61	1.62	1.63	1.64	1.65	1.67	1.68	1.69	1.71	1.72	1.73	1.75	1.76
2.3	1.48	1.49	1.50	1.51	1.52	1.53	1.54	1.55	1.57	1.58	1.59	1.60	1.61	1.62	1.63	1.64	1.65	1.67	1.68	1.69	1.70
2.4	1.45	1.46	1.47	1.48	1.49	1.50	1.51	1.52	1.53	1.54	1.55	1.56	1.57	1.58	1.59	1.60	1.61	1.62	1.63	1.64	1.66
2.5	1.43	1.44	1.44	1.45	1.46	1.47	1.48	1.49	1.50	1.51	1.51	1.52	1.53	1.54	1.55	1.56	1.57	1.58	1.59	1.60	1.61
2.6	1.41	1.41	1.42	1.43	1.44	1.44	1.45	1.46	1.47	1.48	1.49	1.49	1.50	1.51	1.52	1.53	1.54	1.55	1.56	1.57	1.58
2.7	1.39	1.39	1.40	1.41	1.41	1.42	1.43	1.44	1.44	1.45	1.46	1.47	1.47	1.48	1.49	1.50	1.51	1.52	1.53	1.53	1.54
2.8	1.37	1.37	1.38	1.39	1.39	1.40	1.41	1.41	1.42	1.43	1.44	1.44	1.45	1.46	1.47	1.47	1.48	1.49	1.50	1.51	1.51
2.9	1.35	1.36	1.36	1.37	1.37	1.38	1.39	1.39	1.40	1.41	1.42	1.42	1.43	1.44	1.44	1.45	1.46	1.47	1.47	1.48	1.49
3.0	1.33	1.34	1.34	1.35	1.36	1.36	1.37	1.38	1.38	1.39	1.40	1.40	1.41	1.42	1.42	1.43	1.44	1.44	1.45	1.46	1.46
3.5	1.27	1.28	1.28	1.29	1.29	1.30	1.30	1.31	1.31	1.32	1.32	1.33	1.33	1.34	1.34	1.35	1.35	1.36	1.36	1.37	1.37
4.0	1.23	1.23	1.24	1.24	1.25	1.25	1.25	1.26	1.26	1.27	1.27	1.27	1.28	1.28	1.29	1.29	1.29	1.30	1.30	1.31	1.31
4.5	1.20	1.20	1.21	1.21	1.21	1.22	1.22	1.22	1.23	1.23	1.23	1.24	1.24	1.24	1.25	1.25	1.25	1.26	1.26	1.26	1.27
5.0	1.18	1.18	1.18	1.19	1.19	1.19	1.19	1.19	1.20	1.20	1.20	1.21	1.21	1.21	1.22	1.22	1.22	1.22	1.23	1.23	1.23
6.0	1.14	1.15	1.15	1.15	1.15	1.15	1.16	1.16	1.16	1.16	1.16	1.17	1.17	1.17	1.17	1.18	1.18	1.18	1.18	1.18	1.19
7.0	1.12	1.12	1.12	1.13	1.13	1.13	1.13	1.13	1.14	1.14	1.14	1.14	1.14	1.14	1.15	1.15	1.15	1.15	1.15	1.16	1.16
8.0	1.10	1.10	1.11	1.11	1.11	1.11	1.11	1.11	1.12	1.12	1.12	1.12	1.12	1.12	1.13	1.13	1.13	1.13	1.13	1.13	1.14
9.0	1.09	1.09	1.09	1.09	1.10	1.10	1.10	1.10	1.10	1.10	1.11	1.11	1.11	1.11	1.11	1.11	1.11	1.11	1.11	1.12	1.12
10.0	1.08	1.08	1.08	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.09	1.10	1.10	1.10	1.10	1.10	1.10	1.10	1.10
15.0	1.05	1.05	1.05	1.05	1.06	1.06	1.06	1.06	1.06	1.06	1.06	1.06	1.06	1.06	1.06	1.06	1.06	1.07	1.07	1.07	1.07
20.0	1.04	1.04	1.04	1.04	1.04	1.04	1.04	1.04	1.04	1.04	1.04	1.04	1.04	1.05	1.05	1.05	1.05	1.05	1.05	1.05	1.05
25.0	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.04	1.04	1.04	1.04	1.04	1.04	1.04	1.04	1.04	1.04
30.0	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03
35.0	1.02	1.02	1.02	1.02	1.02	1.02	1.02	1.02	1.02	1.02	1.02	1.02	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03	1.03

Examples 6, 7, 8, 17, 18, 19, 20 and 21 refer

For T–sections where stress in stem is neglected j is usually estimated more accurately from Table 8.

Table 10
Coefficients (k) for Rectangular and T–Sections with Various Quantities of Tensile, or Tensile and Compressive Reinforcement



Values of k $= \sqrt{m^2 + 2q} - m$,

where $m = npi^* + p'(2n-1) + p''$ and

$q = npi^* + p'(2n-1)\frac{d'}{d} + p''\frac{t}{2d}$.

Procedure: Compute m and q; select k–value.

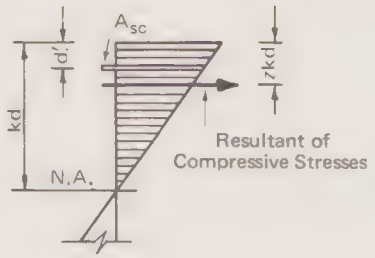
m	q																				
	.03	.04	.05	.06	.07	.08	.09	.10	.11	.12	.13	.14	.15	.16	.17	.18	.19	.20	.21	.22	.23
.06	.192	.229	.262	.292																	
.07	.185	.221	.254	.283	.311																
.08	.178	.214	.246	.376	.303	.328															
.09	.171	.207	.239	.268	.295	.320	.344														
.10	.165	.200	.232	.261	.287	.312	.336	.358													
.11	.159	.193	.225	.253	.280	.305	.328	.351	.372												
.12	.153	.187	.218	.247	.273	.298	.321	.343	.364	.384											
.13	.147	.181	.212	.240	.266	.291	.314	.336	.357	.377	.396										
.14	.142	.176	.206	.234	.260	.284	.307	.329	.349	.370	.389	.407									
.15	.137	.170	.200	.227	.253	.277	.300	.322	.342	.362	.382	.400	.418								
.16	.133	.165	.194	.222	.247	.271	.293	.315	.336	.355	.374	.393	.411	.428							
.17	.128	.160	.189	.216	.241	.265	.287	.308	.329	.349	.367	.386	.404	.421	.437						
.18	.124	.155	.184	.210	.235	.259	.281	.302	.322	.342	.361	.379	.397	.414	.430	.446					
.19	.120	.151	.179	.205	.230	.253	.275	.296	.316	.335	.354	.372	.390	.407	.423	.439	.455				
.20	.116	.146	.174	.200	.224	.247	.269	.290	.310	.329	.348	.366	.383	.400	.416	.432	.448	.463			
.21	.113	.142	.170	.195	.219	.242	.263	.284	.304	.323	.341	.359	.377	.393	.410	.426	.441	.456	.471		
.22	.109	.138	.165	.190	.214	.237	.258	.278	.298	.317	.335	.353	.370	.387	.403	.419	.435	.450	.464	.479	
.23	.106	.135	.161	.186	.209	.231	.253	.273	.292	.311	.329	.347	.364	.381	.397	.413	.428	.443	.458	.472	.486
.24	.103	.131	.157	.181	.205	.226	.247	.268	.287	.306	.324	.341	.358	.374	.391	.406	.422	.436	.451	.465	.479
.26	.097	.124	.149	.173	.196	.217	.238	.257	.276	.295	.312	.330	.346	.363	.378	.394	.409	.424	.438	.452	.466
.28	.092	.118	.142	.165	.187	.208	.228	.248	.266	.284	.302	.319	.335	.351	.367	.382	.397	.412	.426	.440	.454
.30	.087	.112	.136	.158	.180	.200	.220	.239	.257	.274	.292	.308	.325	.340	.356	.371	.386	.400	.414	.428	.442
.32	.083	.107	.130	.152	.172	.192	.211	.230	.248	.265	.282	.298	.314	.330	.345	.360	.375	.389	.403	.416	.430
.34	.079	.102	.124	.145	.166	.185	.204	.222	.239	.256	.273	.289	.305	.320	.335	.350	.364	.378	.392	.405	.419
.36	.075	.098	.119	.140	.159	.178	.196	.214	.231	.248	.264	.280	.295	.311	.325	.340	.354	.368	.381	.395	.408
.38	.072	.094	.114	.134	.153	.172	.190	.207	.224	.240	.256	.271	.287	.301	.316	.330	.344	.358	.371	.384	.397
.42	.066	.086	.106	.124	.142	.160	.177	.194	.210	.225	.241	.256	.270	.285	.299	.312	.326	.339	.352	.365	.378
.46	.061	.080	.098	.116	.133	.150	.166	.182	.197	.212	.227	.241	.255	.269	.283	.296	.309	.322	.335	.347	.360
.50	.057	.074	.092	.108	.125	.140	.156	.171	.186	.200	.214	.228	.242	.255	.268	.281	.294	.306	.319	.331	.343
.54	.053	.070	.086	.102	.117	.132	.147	.161	.175	.189	.203	.216	.229	.242	.255	.267	.280	.292	.304	.315	.327
.58	.050	.065	.081	.096	.110	.125	.139	.152	.166	.179	.192	.205	.218	.230	.242	.255	.266	.278	.290	.301	.312
.62	.047	.061	.076	.090	.104	.118	.131	.144	.157	.170	.183	.195	.207	.219	.231	.243	.254	.266	.277	.288	.299
.66	.044	.058	.072	.085	.099	.112	.125	.137	.150	.162	.174	.186	.198	.209	.221	.232	.243	.254	.265	.276	.286
.70	.042	.055	.068	.081	.094	.106	.119	.131	.143	.154	.166	.178	.189	.200	.211	.222	.233	.243	.254	.264	.275
.74	.039	.052	.065	.077	.089	.101	.113	.125	.136	.147	.159	.170	.181	.191	.202	.213	.223	.233	.244	.254	.264

m	q																				
	.24	.26	.28	.30	.32	.34	.36	.38	.40	.42	.44	.46	.48	.50	.52	.54	.56	.58	.60	.62	.64
.24	.493																				
.26	.480	.507																			
.28	.467	.494	.519																		
.30	.455	.481	.506	.531																	
.32	.443	.469	.494	.518	.542																
.34	.432	.457	.482	.506	.529	.552															
.36	.421	.446	.470	.494	.517	.540	.562														
.38	.410	.435	.459	.483	.506	.528	.550	.571													
.42	.390	.415	.438	.461	.484	.505	.527	.548	.568	.588											
.46	.372	.395	.418	.441	.463	.484	.505	.526	.546	.565	.585	.604									
.50	.354	.378	.400	.422	.443	.464	.485	.505	.525	.544	.563	.582	.600	.618							
.54	.338	.361	.383	.404	.425	.446	.466	.485	.505	.524	.542	.561	.579	.596	.614	.631					
.58	.324	.345	.367	.388	.408	.428	.448	.467	.486	.505	.523	.541	.559	.576	.593	.610	.627	.643			
.62	.310	.331	.352	.372	.392	.412	.431	.450	.468	.487	.504	.522	.539	.557	.573	.590	.607	.623	.639	.655	
.66	.297	.318	.338	.358	.377	.396	.415	.433	.452	.469	.487	.504	.521	.538	.555	.571	.587	.603	.619	.634	.650
.70	.285	.305	.325	.344	.363	.382	.400	.418	.436	.453	.470	.487	.504	.521	.537	.553	.569	.585	.600	.615	.630
.74	.274	.293	.312	.331	.350	.368	.386	.404	.421	.438	.455	.471	.488	.504	.520	.536	.551	.567	.582	.597	.612
.78	.263	.282	.301	.319	.337	.355	.373	.390	.407	.423	.440	.456	.472	.488	.504	.519	.535	.550	.565	.580	.594
.82	.253	.272	.290	.308	.326	.343	.360	.377	.393	.410	.426	.442	.458	.473	.489	.504	.519	.534	.548	.563	.577
.86	.244	.262	.280	.297	.315	.331	.348	.365	.381	.397	.413	.428	.444	.459	.474	.489	.504	.518	.533	.547	.561
.90	.236	.253	.270	.287	.304	.321	.337	.353	.369	.385	.400	.415	.430	.445	.460	.475	.489	.504	.518	.532	.546
.94	.228	.245	.261	.278	.294	.310	.326	.342	.358	.373	.388	.403	.418	.432	.447	.461	.475	.490	.503	.517	.531
.98	.220	.237	.253	.269	.285	.301	.316	.332	.347	.362	.377	.391	.406	.420	.434	.448	.462	.476	.490	.503	.517
1.02	.213	.229	.245	.261	.276	.292	.307	.322	.337	.351	.366	.380	.394	.408	.422	.436	.450	.463	.477	.490	.503

Examples 12, 13, 15, 16, 17, 18, 20, 22 and 23 refer

* i = 1 in the case of simple bending, and may be obtained from Table 9 in the case of combined bending and axial compression.

Table 11 *
Coefficients (z) for Position of Resultant of Compressive Stresses in Rectangular Sections



$$\text{Values of } z = \frac{\frac{1}{6} + \frac{(2n-1)A_{sc}}{kbd} \times \frac{d'}{kd} \times \left(1 - \frac{d'}{kd}\right)}{\frac{1}{2} + \frac{(2n-1)A_{sc}}{kbd} \times \left(1 - \frac{d'}{kd}\right)}$$

Enter table with known values of

$\frac{1}{k} \times \frac{(2n-1)A_{sc}}{bd}$ and $\frac{1}{k} \times \frac{d'}{d}$; select z.

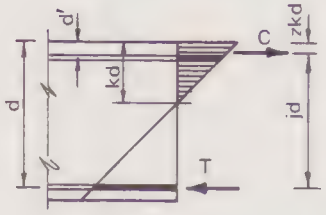
Compute $j = 1 - zk$ (See Table 12)

$\frac{1}{k} \times \frac{d'}{d}$	$\frac{1}{k} \times \frac{(2n-1)A_{sc}}{bd}$																				
	0	.05	.10	.15	.20	.25	.30	.35	.40	.45	.50	.55	.60	.65	.70	.75	.80	.90	1.0	1.5	2.0
.02	.33	.31	.28	.26	.24	.23	.22	.21	.20	.19	.18	.17	.16	.16	.15	.15	.14	.13	.13	.10	.08
.04	.33	.31	.29	.27	.25	.24	.23	.22	.21	.20	.19	.18	.18	.17	.17	.16	.16	.15	.14	.12	.10
.06	.33	.31	.29	.27	.26	.25	.24	.23	.22	.21	.20	.19	.19	.18	.18	.17	.17	.16	.16	.13	.12
.08	.33	.31	.29	.28	.26	.25	.24	.23	.23	.22	.21	.21	.20	.20	.19	.19	.18	.18	.17	.15	.13
.10	.33	.31	.30	.28	.27	.26	.25	.24	.24	.23	.22	.22	.21	.21	.20	.20	.20	.19	.18	.16	.15
.12	.33	.32	.30	.29	.28	.27	.26	.25	.25	.24	.23	.23	.22	.22	.22	.21	.21	.20	.20	.18	.17
.14	.33	.32	.31	.29	.28	.28	.27	.26	.25	.25	.24	.24	.24	.23	.23	.22	.22	.22	.21	.19	.18
.16	.33	.32	.31	.30	.29	.28	.28	.27	.26	.26	.25	.25	.25	.24	.24	.24	.23	.23	.22	.21	.20
.18	.33	.32	.31	.30	.30	.29	.28	.28	.27	.27	.26	.26	.26	.25	.25	.25	.25	.24	.24	.22	.22
.20	.33	.32	.32	.31	.30	.30	.29	.29	.28	.28	.27	.27	.27	.27	.26	.26	.26	.25	.25	.24	.23
.22	.33	.32	32	.31	.31	.30	.30	.29	.29	.29	.28	.28	.28	.28	.27	.27	.27	.27	.26	.25	.25
.24	.33	.33	.32	.32	.31	.31	.30	.30	.30	.30	.29	.29	.29	.29	.28	.28	.28	.28	.28	.27	.26
.26	.33	.33	.32	.32	.32	.31	.31	.31	.31	.30	.30	.30	.30	.30	.30	.29	.29	.29	.29	.28	.28
.28	.33	.33	.33	.32	.32	.32	.32	.32	.31	.31	.31	.31	.31	.31	.31	.31	.30	.30	.30	.30	.29
.30	.33	.33	.33	.33	.33	.32	.32	.32	.32	.32	.32	.32	.32	.32	.32	.32	.32	.32	.31	.31	.31
.32	.33	.33	.33	.33	.33	.33	.33	.33	.33	.33	.33	.33	.33	.33	.33	.33	.33	.33	.33	.32	.32
.34	.33	.33	.33	.33	.34	.34	.34	.34	.34	.34	.34	.34	.34	.34	.34	.34	.34	.34	.34	.34	.34
.36	.33	.33	.34	.34	.34	.34	.34	.34	.34	.34	.34	.34	.34	.35	.35	.35	.35	.35	.35	.36	.35
.38	.33	.34	.34	.34	.34	.35	.35	.35	.35	.35	.35	.35	.35	.35	.36	.36	.36	.36	.36	.36	.37
.40	.33	.34	.34	.34	.35	.35	.35	.35	.36	.36	.36	.36	.36	.36	.36	.37	.37	.37	.37	.38	.38
.42	.33	.34	.34	.35	.35	.35	.36	.36	.36	.36	.37	.37	.37	.37	.37	.37	.38	.38	.38	.39	.39
.44	.33	.34	.34	.35	.35	.36	.36	.36	.37	.37	.37	.37	.38	.38	.38	.38	.38	.39	.39	.40	.41
.46	.33	.34	.35	.35	.36	.36	.36	.37	.37	.37	.38	.38	.38	.39	.39	.39	.39	.40	.40	.41	.42
.48	.33	.34	.35	.35	.36	.36	.37	.37	.38	.38	.38	.39	.39	.39	.39	.40	.40	.40	.41	.42	.43
.50	.33	.34	.35	.35	.36	.37	.37	.38	.38	.38	.39	.39	.40	.40	.40	.40	.41	.41	.42	.43	.44

Examples 12, 13, 15, 18, 20 and 23 refer

* This table can also be used to find coefficient (z) for T–sections with tensile reinforcement only taking into account the compressive stress in the stem. This is done by substituting $\frac{(b-b_w)t}{b_w d}$ for $\frac{(2n-1)A_{sc}}{bd}$ and $\frac{yt}{d}$ for $\frac{d'}{d}$ as illustrated in Examples 15 and 20.

Table 12
Coefficients (j) for Rectangular Sections with or without Compressive Reinforcement



Values of $j = 1 - zk$

For $\frac{d'}{d} = \frac{k}{3}$ or $A_{sc} = 0$: $z = \frac{1}{3}$ and $j = 1 - \frac{k}{3}$.

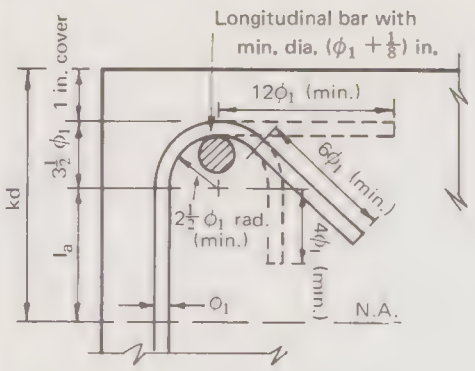
z	k																				
	.20	.22	.24	.26	.28	.30	.32	.34	.36	.38	.40	.42	.44	.46	.48	.50	.52	.54	.56	.58	.60
.20	.96	.96	.95	.95	.94	.94	.94	.93	.93	.92	.92	.92	.91	.91	.90	.90	.90	.89	.89	.88	.88
.21	.96	.95	.95	.95	.94	.94	.93	.93	.92	.92	.92	.91	.91	.90	.90	.90	.89	.89	.88	.88	.87
.22	.96	.95	.95	.94	.94	.93	.93	.93	.92	.92	.91	.91	.90	.90	.89	.89	.89	.88	.88	.87	.87
.23	.95	.95	.94	.94	.94	.93	.93	.92	.92	.91	.91	.90	.90	.89	.89	.89	.88	.88	.87	.87	.86
.24	.95	.95	.94	.94	.93	.93	.92	.92	.91	.91	.90	.90	.89	.89	.88	.88	.88	.87	.87	.86	.86
.25	.95	.95	.94	.94	.93	.93	.92	.92	.91	.91	.90	.90	.89	.89	.88	.88	.87	.87	.86	.86	.85
.26	.95	.94	.94	.93	.93	.92	.92	.91	.91	.90	.90	.89	.89	.88	.88	.87	.86	.86	.85	.85	.84
.27	.95	.94	.94	.93	.92	.92	.91	.91	.90	.90	.89	.89	.88	.88	.87	.87	.86	.85	.85	.84	.84
.28	.94	.94	.93	.93	.92	.92	.91	.90	.90	.89	.89	.88	.88	.87	.87	.86	.85	.85	.84	.84	.83
.29	.94	.94	.93	.92	.92	.91	.91	.90	.90	.89	.88	.88	.87	.87	.86	.86	.85	.84	.84	.83	.83
.30	.94	.93	.93	.92	.92	.91	.90	.90	.89	.89	.88	.87	.87	.86	.86	.85	.84	.84	.83	.83	.82
.31	.94	.93	.93	.92	.91	.91	.90	.89	.89	.88	.88	.87	.86	.86	.85	.85	.84	.83	.83	.82	.81
.32	.94	.93	.92	.92	.91	.90	.90	.89	.88	.88	.87	.87	.86	.85	.85	.84	.83	.83	.82	.81	.81
.33	.93	.93	.92	.91	.91	.90	.89	.89	.88	.87	.87	.86	.85	.85	.84	.84	.83	.82	.82	.81	.80
.34	.93	.93	.92	.91	.90	.90	.89	.88	.88	.87	.86	.86	.85	.84	.84	.83	.82	.82	.81	.80	.80
.35	.93	.92	.92	.91	.90	.90	.89	.88	.87	.87	.86	.85	.85	.84	.83	.83	.82	.81	.80	.80	.79
.36	.93	.92	.91	.91	.90	.89	.88	.88	.87	.86	.86	.85	.84	.83	.83	.82	.81	.81	.80	.79	.78
.37	.93	.92	.91	.90	.90	.89	.88	.87	.87	.86	.85	.84	.84	.83	.82	.82	.81	.80	.79	.79	.78
.38	.92	.92	.91	.90	.89	.89	.88	.87	.86	.86	.85	.84	.83	.83	.82	.81	.80	.79	.79	.78	.77
.39	.92	.91	.91	.90	.89	.88	.88	.87	.86	.85	.84	.84	.83	.82	.81	.81	.80	.79	.78	.77	.77

Examples 12, 13, 15, 17, 18, 20, 22 and 23 refer

Table 13

Values of $\frac{A_{sv}\ F_{sv}}{1{,}000}$ for Hooked Stirrups as Transverse Shear Reinforcement

$\phi 2$



Values of $\frac{A_{sv}\ F_{sv}}{1{,}000} = \frac{A_{sv}}{1{,}000}\left(H_a + \frac{4\ U_{av} \times l_a}{\phi_1}\right)$(kips),

for $\left(H_a + \frac{4\ U_{av} \times l_a}{\phi_1}\right)$ not greater than F_{sv} = 20,000 psi.,

where:

hook allowance H_a* = the lesser of 12,000 psi. or $6F'_c$;

embedded length l_a = $(kd - 3\frac{1}{2}\ \phi_1 - 1)$ in.;

ϕ_1 = stirrup bar diameter (in.);

A_{sv} = area of 2 vertical legs of stirrup = $\frac{\pi\ \phi_1^2}{2}$

Values of k from Table 1 and of U_{av} from CA2–1963.

Stirrup bar size: $\phi 2$

Effective depth d (in.)	F'_c = 2,000				F'_c = 2,500				F'_c = 3,000				F'_c = 3,500				F'_c = 4,000				F'_c = 5,000		
	F_s Tensile stress in main bars (ksi.)																						
	15	20	25	30	15	20	25	30	15	20	25	30	15	20	25	30	15	20	25	30	20	25	30
9	1.48	1.40	1.34	1.29	1.54	1.45	1.38	1.33	1.62	1.52	1.44	1.38	1.69	1.58	1.49	1.42	1.75	1.63	1.54	1.46	1.68	1.58	1.51
9.5	1.51	1.42	1.36	1.31	1.58	1.48	1.41	1.35	1.66	1.55	1.47	1.41	1.73	1.62	1.53	1.46	1.80	1.67	1.57	1.50	1.73	1.63	1.55
10	1.54	1.45	1.38	1.33	1.61	1.51	1.44	1.38	1.70	1.59	1.50	1.44	1.78	1.66	1.56	1.49	1.85	1.72	1.61	1.53	1.77	1.67	1.58
10.5	1.57	1.47	1.41	1.35	1.65	1.54	1.46	1.40	1.74	1.62	1.53	1.46	1.83	1.70	1.60	1.52	1.90	1.76	1.65	1.57	1.82	1.71	1.62
11	1.60	1.50	1.43	1.37	1.68	1.57	1.49	1.42	1.78	1.66	1.56	1.49	1.87	1.74	1.63	1.55	1.95	1.81	1.69	1.60	1.87	1.75	1.66
11.5	1.63	1.53	1.45	1.39	1.72	1.60	1.51	1.45	1.82	1.69	1.59	1.52	1.92	1.78	1.67	1.58	1.96	1.85	1.73	1.64	1.92	1.80	1.70
12	1.66	1.55	1.47	1.41	1.75	1.63	1.54	1.47	1.86	1.73	1.62	1.55	1.96	1.82	1.70	1.61		1.90	1.77	1.67	1.96	1.84	1.74
12.5	1.69	1.58	1.49	1.43	1.79	1.66	1.57	1.49	1.90	1.76	1.66	1.57		1.86	1.74	1.65		1.94	1.81	1.71		1.88	1.77
13	1.72	1.60	1.52	1.45	1.82	1.69	1.59	1.52	1.94	1.80	1.69	1.60		1.90	1.77	1.68		1.96	1.85	1.74		1.92	1.81
13.5	1.75	1.63	1.54	1.47	1.86	1.72	1.62	1.54	1.96	1.83	1.72	1.63		1.94	1.81	1.71			1.89	1.78		1.96	1.85
14	1.78	1.65	1.56	1.49	1.89	1.75	1.64	1.56		1.87	1.75	1.66		1.96	1.84	1.74			1.93	1.81			1.89
14.5	1.81	1.68	1.58	1.51	1.93	1.78	1.67	1.59		1.90	1.78	1.68			1.88	1.77			1.96	1.85			1.93
15	1.84	1.70	1.60	1.53	1.96	1.81	1.70	1.61		1.94	1.81	1.71			1.91	1.80				1.88			1.96
15.5	1.87	1.73	1.63	1.55		1.84	1.72	1.63		1.96	1.84	1.74			1.95	1.83				1.92			
16	1.90	1.75	1.65	1.57		1.87	1.75	1.66			1.87	1.77			1.96	1.87				1.96			
16.5	1.93	1.78	1.67	1.59		1.90	1.77	1.68			1.90	1.79				1.90							
17	1.96	1.80	1.69	1.61		1.93	1.80	1.70			1.93	1.82				1.93							
17.5		1.83	1.71	1.62		1.96	1.83	1.72			1.96	1.85				1.96							
18		1.85	1.74	1.64			1.85	1.75				1.88											
19		1.91	1.78	1.68			1.90	1.79				1.93											
20		1.96	1.83	1.72			1.96	1.84				1.96											
21			1.87	1.76				1.89															
22			1.91	1.80				1.93															
23			1.96	1.84				1.96															
24				1.88																			
25				1.92																			
26				1.96																			
27																							
28																							
29																							

Examples 10 and 11 refer

continued

* The hook allowance H_a = the lesser of 12,000 psi. or $6F'_c$, for a standard hook only as defined in AS CA2–1963, which embraces a longitudinal bar of diameter not less than $(\phi_1 + \frac{1}{8})$ in. In addition the leg of the stirrup must be perpendicular to the longitudinal bar.

Values of $\frac{A_{sv} F_{sv}}{1{,}000}$ for stirrup bar size ϕ3

Effective depth d (in.)	F'_c = 2,000				F'_c = 2,500				F'_c = 3,000				F'_c = 3,500				F'_c = 4,000				F'_c = 5,000		
	F_s = Tensile stress in main bars (ksi.)																						
	15	20	25	30	15	20	25	30	15	20	25	30	15	20	25	30	15	20	25	30	20	25	30
9	3.02	2.90	2.81	2.74	3.10	2.97	2.87	2.79	3.21	3.06	2.94	2.85	3.30	3.13	3.01	2.91	3.38	3.20	3.06	2.95	3.28	3.14	3.02
9.5	3.06	2.94	2.84	2.77	3.16	3.01	2.90	2.82	3.27	3.11	2.99	2.89	3.37	3.19	3.06	2.95	3.46	3.27	3.12	3.00	3.35	3.20	3.08
10	3.11	2.97	2.88	2.80	3.21	3.06	2.94	2.86	3.33	3.16	3.03	2.93	3.44	3.25	3.11	3.00	3.54	3.34	3.18	3.06	3.42	3.26	3.14
10.5	3.15	3.01	2.91	2.83	3.26	3.10	2.98	2.89	3.39	3.21	3.08	2.97	3.51	3.31	3.16	3.05	3.61	3.40	3.24	3.11	3.49	3.33	3.19
11	3.20	3.05	2.94	2.86	3.31	3.15	3.02	2.93	3.45	3.27	3.12	3.02	3.58	3.37	3.22	3.10	3.69	3.47	3.30	3.16	3.56	3.39	3.25
11.5	3.24	3.09	2.98	2.89	3.37	3.19	3.06	2.96	3.51	3.32	3.17	3.06	3.65	3.43	3.27	3.14	3.77	3.54	3.36	3.22	3.63	3.45	3.31
12	3.29	3.13	3.01	2.92	3.42	3.23	3.10	3.00	3.57	3.37	3.22	3.10	3.72	3.49	3.32	3.19	3.85	3.61	3.42	3.27	3.70	3.52	3.36
12.5	3.33	3.16	3.04	2.95	3.47	3.28	3.14	3.03	3.63	3.43	3.26	3.14	3.79	3.56	3.38	3.24	3.92	3.67	3.48	3.32	3.77	3.58	3.42
13	3.38	3.20	3.08	2.97	3.52	3.32	3.18	3.07	3.70	3.48	3.31	3.18	3.86	3.62	3.43	3.29	4.00	3.74	3.54	3.38	3.85	3.64	3.48
13.5	3.42	3.24	3.11	3.00	3.58	3.37	3.22	3.10	3.76	3.53	3.36	3.22	3.93	3.68	3.48	3.33	4.08	3.81	3.60	3.43	3.92	3.71	3.54
14	3.47	3.28	3.14	3.03	3.63	3.41	3.26	3.13	3.82	3.58	3.40	3.26	4.00	3.74	3.54	3.38	4.15	3.87	3.66	3.48	3.99	3.77	3.59
14.5	3.51	3.32	3.17	30.6	3.68	3.46	3.30	3.17	3.88	3.64	3.45	3.31	4.07	3.80	3.59	3.43	4.23	3.94	3.71	3.53	4.06	3.83	3.65
15	3.56	3.35	3.21	3.09	3.73	3.50	3.33	3.20	3.94	3.69	3.50	3.35	4.14	3.86	3.64	3.47	4.31	4.01	3.77	3.59	4.13	3.90	3.71
15.5	3.60	3.39	3.24	3.12	3.78	3.55	3.37	3.24	4.00	3.74	3.54	3.39	4.21	3.92	3.69	3.52	4.39	4.07	3.83	3.64	4.20	3.96	3.76
16	3.64	3.43	3.27	3.15	3.84	3.59	3.41	3.27	4.06	3.80	3.59	3.43	4.27	3.98	3.75	3.57	4.42	4.14	3.89	3.69	4.27	4.02	3.82
16.5	3.69	3.47	3.31	3.18	3.89	3.64	3.45	3.31	4.12	3.85	3.63	3.47	4.34	4.04	3.80	3.62		4.21	3.95	3.75	4.34	4.09	3.88
17	3.73	3.51	3.34	3.21	3.94	3.68	3.49	3.34	4.19	3.90	3.68	3.51	4.41	4.10	3.85	3.66		4.28	4.01	3.80	4.41	4.15	3.93
17.5	3.78	3.54	3.37	3.24	3.99	3.73	3.53	3.38	4.25	3.95	3.73	3.55	4.42	4.16	3.91	3.71		4.34	4.07	3.85	4.42	4.21	3.99
18	3.82	3.58	3.41	3.27	4.05	3.77	3.57	3.41	4.31	4.01	3.77	3.59		4.22	3.96	3.76		4.41	4.13	3.91		4.28	4.05
19	3.91	3.66	3.47	3.33	4.15	3.86	3.65	3.48	4.42	4.11	3.87	3.68		4.34	4.07	3.85		4.42	4.25	4.01		4.40	4.16
20	4.00	3.73	3.54	3.38	4.26	3.95	3.73	3.55		4.22	3.96	3.76		4.42	4.17	3.95			4.37	4.12		4.42	4.28
21	4.09	3.81	3.60	3.44	4.36	4.04	3.80	3.62		4.32	4.05	3.84			4.28	4.04			4.42	4.22			4.39
22	4.18	3.89	3.67	3.50	4.42	4.13	3.88	3.69		4.42	4.14	3.93			4.38	4.14				4.33			4.42
23	4.27	3.96	3.74	3.56		4.22	3.96	3.76			4.24	4.01			4.42	4.23				4.42			
24	4.36	4.04	3.80	3.62		4.31	4.04	3.83			4.33	4.09				4.33							
25	4.42	4.11	3.87	3.68		4.40	4.12	3.90			4.42	4.17				4.42							
26		4.19	3.94	3.73		4.42	4.20	3.97				4.26											
27		4.27	4.00	3.79			4.27	4.04				4.34											
28		4.34	4.07	3.85			4.35	4.11				4.42											
29		4.42	4.13	3.91			4.42	4.18															
30			4.20	3.97				4.25															
31			4.27	4.03				4.32															
32			4.33	4.09				4.39															
33			4.40	4.14				4.42															
34			4.42	4.20																			
36				4.32																			
38				4.42																			
40																							
42																							
44																							

continued

Values of $\frac{A_{sv}\ F_{sv}}{1{,}000}$ for stirrup bar size ϕ4

Effective depth d (in.)	F'_c = 2,000				F'_c = 2,500				F'_c = 3,000				F'_c = 3,500				F'_c = 4,000				F'_c = 5,000		
	F_s = Tensile stress in main bars (ksi.)																						
	15	20	25	30	15	20	25	30	15	20	25	30	15	20	25	30	15	20	25	30	20	25	30
9	5.09	4.93	4.82	4.72	5.19	5.01	4.87	4.77	5.32	5.12	4.96	4.84	5.43	5.21	5.03	4.90	5.52	5.28	5.10	4.95	5.38	5.20	5.04
9.5	5.15	4.98	4.86	4.76	5.26	5.07	4.93	4.82	5.40	5.19	5.02	4.90	5.52	5.29	5.11	4.96	5.63	5.37	5.18	5.02	5.48	5.28	5.12
10	5.21	5.03	4.90	4.80	5.33	5.13	4.98	4.86	5.48	5.26	5.08	4.95	5.61	5.37	5.18	5.03	5.73	5.46	5.26	5.09	5.57	5.36	5.19
10.5	5.27	5.09	4.95	4.84	5.40	5.19	5.03	4.91	5.56	5.33	5.15	5.01	5.71	5.45	5.25	5.09	5.83	5.55	5.33	5.16	5.67	5.45	5.27
11	5.33	5.14	4.99	4.88	5.47	5.25	5.08	4.96	5.64	5.40	5.21	5.06	5.80	5.53	5.32	5.15	5.94	5.64	5.41	5.23	5.76	5.53	5.35
11.5	5.39	5.19	5.04	4.92	5.54	5.31	5.14	5.00	5.72	5.47	5.27	5.12	5.89	5.61	5.39	5.22	6.04	5.73	5.49	5.30	5.86	5.62	5.42
12	5.45	5.24	5.08	4.96	5.61	5.37	5.19	5.05	5.81	5.54	5.33	5.17	5.99	5.69	5.46	5.28	6.14	5.82	5.57	5.37	5.95	5.70	5.50
12.5	5.51	5.29	5.12	5.00	5.68	5.43	5.24	5.09	5.89	5.61	5.39	5.23	6.08	5.77	5.53	5.34	6.24	5.91	5.65	5.44	6.05	5.79	5.57
13	5.57	5.34	5.17	5.03	5.75	5.49	5.29	5.14	5.97	5.68	5.45	5.28	6.17	5.85	5.60	5.41	6.35	6.00	5.73	5.51	6.14	5.87	5.65
13.5	5.63	5.39	5.21	5.07	5.82	5.55	5.34	5.19	6.05	5.75	5.52	5.34	6.26	5.93	5.67	5.47	6.45	6.09	5.81	5.58	6.24	5.96	5.73
14	5.69	5.44	5.26	5.11	5.89	5.61	5.40	5.23	6.13	5.82	5.58	5.39	6.36	6.01	5.74	5.53	6.55	6.18	5.89	5.65	6.33	6.04	5.80
14.5	5.75	5.49	5.30	5.15	5.96	5.67	5.45	5.28	6.21	5.89	5.64	5.45	6.45	6.09	5.81	5.60	6.65	6.27	5.97	5.73	6.43	6.12	5.88
15	5.81	5.54	5.34	5.19	6.03	5.73	5.50	5.33	6.29	5.96	5.70	5.50	6.54	6.17	5.88	5.66	6.76	6.36	6.05	5.80	6.52	6.21	5.95
15.5	5.87	5.59	5.39	5.23	6.10	5.78	5.55	5.37	6.38	6.03	5.76	5.56	6.63	6.25	5.95	5.72	6.86	6.45	6.12	5.87	6.61	6.29	6.03
16	5.93	5.64	5.43	5.27	6.17	5.84	5.60	5.42	6.46	6.10	5.82	5.61	6.73	6.33	6.02	5.79	6.96	6.53	6.20	5.94	6.71	6.38	6.11
16.5	5.99	5.69	5.48	5.31	6.24	5.90	5.66	5.47	6.54	6.17	5.89	5.67	6.82	6.41	6.09	5.85	7.07	6.62	6.28	6.01	6.80	6.46	6.18
17	6.05	5.74	5.52	5.35	6.31	5.96	5.71	5.51	6.62	6.24	5.95	5.72	6.91	6.49	6.17	5.91	7.17	6.71	6.36	6.08	6.90	6.55	6.26
17.5	6.11	5.79	5.57	5.39	6.38	6.02	5.76	5.56	6.70	6.31	6.01	5.78	7.00	6.57	6.24	5.98	7.27	6.80	6.44	6.15	6.99	6.63	6.33
18	6.17	5.84	5.61	5.42	6.45	6.08	5.81	5.60	6.78	6.38	6.07	5.83	7.10	6.65	6.31	6.04	7.37	6.89	6.52	6.22	7.09	6.72	6.41
19	6.29	5.95	5.70	5.50	6.59	6.20	5.92	5.70	6.95	6.52	6.19	5.94	7.28	6.81	6.45	6.17	7.58	7.07	6.68	6.36	7.28	6.88	6.56
20	6.40	6.05	5.79	5.58	6.73	6.32	6.02	5.79	7.11	6.66	6.32	6.05	7.47	6.97	6.59	6.29	7.78	7.25	6.83	6.50	7.47	7.05	6.71
21	6.52	6.15	5.87	5.66	6.87	6.44	6.13	5.88	7.27	6.80	6.44	6.16	7.65	7.13	6.73	6.42	7.85	7.43	6.99	6.64	7.66	7.22	6.87
22	6.64	6.25	5.96	5.74	7.01	6.56	6.23	5.98	7.44	6.95	6.57	6.27	7.84	7.29	6.87	6.55		7.61	7.15	6.79	7.85	7.39	7.02
23	6.76	6.35	6.05	5.81	7.15	6.68	6.33	6.07	7.60	7.09	6.69	6.39	7.85	7.45	7.01	6.67		7.79	7.31	6.93		7.56	7.17
24	6.88	6.45	6.14	5.89	7.29	6.80	6.44	6.16	7.76	7.23	6.81	6.50		7.61	7.15	6.80		7.85	7.47	7.07		7.73	7.32
25	7.00	6.55	6.23	5.97	7.43	6.92	6.54	6.25	7.85	7.37	6.94	6.61		7.77	7.30	6.92			7.62	7.21		7.85	7.47
26	7.12	6.66	6.32	6.05	7.57	7.04	6.65	6.34		7.51	7.06	6.72		7.85	7.44	7.05			7.78	7.35			7.63
27	7.24	6.76	6.40	6.13	7.71	7.16	6.75	6.44		7.65	7.18	6.83			7.58	7.18			7.85	7.49			7.78
28	7.36	6.86	6.49	6.20	7.85	7.28	6.86	6.53		7.79	7.31	6.94			7.72	7.30				7.63			7.85
29	7.48	6.96	6.58	6.28		7.40	6.96	6.62		7.85	7.43	7.05			7.85	7.43				7.78			
30	7.60	7.06	6.67	6.36		7.51	7.07	6.72			7.55	7.16				7.56				7.85			
31	7.71	7.16	6.76	6.44		7.63	7.17	6.81			7.68	7.27				7.68							
32	7.83	7.26	6.84	6.51		7.75	7.27	6.90			7.80	7.38				7.81							
33	7.85	7.36	6.93	6.59		7.85	7.38	7.00			7.85	7.49				7.85							
34		7.47	7.02	6.67			7.48	7.09				7.60											
36		7.67	7.20	6.83			7.69	7.27				7.82											
38		7.85	7.37	6.98			7.85	7.46				7.85											
40			7.55	7.14				7.64															
42			7.73	7.29				7.83															
44			7.85	7.45				7.85															
46				7.61																			
48				7.76																			
50				7.85																			
52																							
54																							

continued

Values of $\frac{A_{sv} F_{sv}}{1{,}000}$ for stirrup bar size #3

Effective depth d (in.)	F'_c = 2,000				F'_c = 2,500				F'_c = 3,000				F'_c = 3,500				F'_c = 4,000				F'_c = 5,000		
	F_s = Tensile stress in main bars (ksi.)																						
	15	20	25	30	15	20	25	30	15	20	25	30	15	20	25	30	15	20	25	30	20	25	30
9	3.39	3.15	2.97	2.83	3.56	3.28	3.08	2.92	3.76	3.46	3.23	3.05	3.95	3.62	3.36	3.16	4.12	3.76	3.48	3.25	3.90	3.62	3.39
9.5	3.48	3.22	3.04	2.89	3.66	3.37	3.16	2.99	3.88	3.57	3.32	3.13	4.09	3.74	3.47	3.25	4.27	3.89	3.59	3.36	4.05	3.75	3.51
10	3.57	3.30	3.10	2.95	3.77	3.46	3.24	3.06	4.01	3.67	3.41	3.22	4.23	3.86	3.57	3.35	4.42	4.02	3.71	3.46	4.19	3.88	3.62
10.5	3.66	3.37	3.17	3.01	3.87	3.55	3.31	3.13	4.13	3.78	3.51	3.30	4.37	3.98	3.68	3.44		4.16	3.83	3.57	4.33	4.00	3.74
11	3.75	3.45	3.23	3.06	3.98	3.64	3.39	3.20	4.25	3.88	3.60	3.38	4.42	4.10	3.78	3.54		4.29	3.95	3.68	4.42	4.13	3.85
11.5	3.83	3.53	3.30	3.12	4.08	3.73	3.47	3.27	4.37	3.99	3.69	3.46		4.22	3.89	3.63		4.42	4.07	3.78		4.26	3.96
12	3.92	3.60	3.37	3.18	4.18	3.82	3.55	3.34	4.42	4.09	3.78	3.55		4.34	4.00	3.73			4.19	3.89		4.38	4.08
12.5	4.01	3.68	3.43	3.24	4.29	3.91	3.63	3.41		4.20	3.88	3.63		4.42	4.10	3.82			4.30	3.99		4.42	4.19
13	4.10	3.75	3.50	3.30	4.39	4.00	3.70	3.48		4.31	3.97	3.71			4.21	3.92			4.42	4.10			4.31
13.5	4.19	3.83	3.57	3.36	4.42	4.09	3.78	3.55		4.41	4.06	3.79			4.31	4.01				4.21			4.42
14	4.28	3.91	3.63	3.42		4.18	3.86	3.62		4.42	4.15	3.88			4.42	4.11				4.31			
14.5	4.37	3.98	3.70	3.47		4.27	3.94	3.69			4.25	3.96				4.20				4.42			
15	4.42	4.06	3.76	3.53		4.36	4.02	3.76			4.34	4.04				4.30							
15.5		4.13	3.83	3.59		4.42	4.10	3.83			4.42	4.13				4.39							
16		4.21	3.90	3.65			4.17	3.90				4.21				4.42							
16.5		4.29	3.96	3.71			4.25	3.97				4.29											
17		4.36	4.03	3.77			4.33	4.04				4.37											
17.5		4.42	4.10	3.82			4.41	4.10				4.42											
18			4.16	3.88			4.42	4.17															
19			4.29	4.00				4.31															
20			4.42	4.12				4.42															
21				4.23																			
22				4.35																			
23				4.42																			
24																							

continued

Values of $\frac{A_{sv}\ F_{sv}}{1{,}000}$ for stirrup bar size #4

Effective depth d (in.)	F'_c = 2,000				F'_c = 2,500				F'_c = 3,000				F'_c = 3,500				F'_c = 4,000				F'_c = 5,000		
	F_s = Tensile stress in main bars (ksi.)																						
	15	20	25	30	15	20	25	30	15	20	25	30	15	20	25	30	15	20	25	30	20	25	30
9	5.47	5.15	4.92	4.73	5.67	5.31	5.04	4.83	5.92	5.52	5.21	4.97	6.15	5.70	5.36	5.09	6.34	5.86	5.48	5.18	6.05	5.68	5.37
9.5	5.59	5.25	5.01	4.81	5.81	5.42	5.14	4.92	6.08	5.66	5.33	5.08	6.33	5.86	5.50	5.22	6.54	6.03	5.64	5.32	6.24	5.85	5.53
10	5.71	5.36	5.09	4.89	5.95	5.54	5.24	5.01	6.25	5.80	5.45	5.19	6.52	6.02	5.64	5.34	6.75	6.21	5.80	5.47	6.43	6.02	5.68
10.5	5.83	5.46	5.18	4.97	6.09	5.66	5.35	5.11	6.41	5.94	5.58	5.30	6.70	6.18	5.78	5.47	6.95	6.39	5.96	5.61	6.62	6.19	5.83
11	5.95	5.56	5.27	5.04	6.23	5.78	5.45	5.20	6.57	6.08	5.70	5.41	6.89	6.34	5.92	5.59	7.16	6.57	6.11	5.75	6.81	6.35	5.98
11.5	6.07	5.66	5.36	5.12	6.37	5.90	5.56	5.29	6.74	6.22	5.82	5.52	7.07	6.50	6.06	5.72	7.36	6.75	6.27	5.89	7.00	6.52	6.13
12	6.19	5.76	5.45	5.20	6.51	6.02	5.66	5.38	6.90	6.36	5.95	5.63	7.26	6.66	6.20	5.85	7.57	6.93	6.43	6.03	7.19	6.69	6.29
12.5	6.31	5.86	5.54	5.28	6.65	6.14	5.77	5.48	7.06	6.50	6.07	5.74	7.44	6.82	6.35	5.97	7.78	7.11	6.59	6.17	7.38	6.86	6.44
13	6.43	5.96	5.62	5.36	6.79	6.26	5.87	5.57	7.22	6.64	6.20	5.85	7.63	6.98	6.49	6.10	7.85	7.28	6.75	6.31	7.57	7.03	6.59
13.5	6.55	6.07	5.71	5.43	6.93	6.38	5.97	5.66	7.39	6.79	6.32	5.96	7.81	7.14	6.63	6.23		7.46	6.90	6.46	7.76	7.20	6.74
14	6.67	6.17	5.80	5.51	7.07	6.50	6.08	5.75	7.55	6.93	6.44	6.07	7.85	7.30	6.77	6.35		7.64	7.06	6.60	7.85	7.37	6.89
14.5	6.79	6.27	5.89	5.59	7.21	6.62	6.18	5.85	7.71	7.07	6.57	6.18		7.46	6.91	6.48		7.82	7.22	6.74		7.54	7.04
15	6.90	6.37	5.98	5.67	7.35	6.74	6.29	5.94	7.85	7.21	6.69	6.29		7.62	7.05	6.61		7.85	7.38	6.88		7.71	7.20
15.5	7.02	6.47	6.07	5.75	7.49	6.86	6.39	6.03		7.35	6.81	6.40		7.78	7.19	6.73			7.54	7.02		7.85	7.35
16	7.14	6.57	6.15	5.82	7.63	6.98	6.50	6.13		7.49	6.94	6.51		7.85	7.34	6.86			7.69	7.16			7.50
16.5	7.26	6.67	6.24	5.90	7.77	7.10	6.60	6.22		7.63	7.06	6.62			7.48	6.99			7.85	7.30			7.65
17	7.38	6.77	6.33	5.98	7.85	7.21	6.71	6.31		7.77	7.18	6.73			7.62	7.11				7.45			7.80
17.5	7.50	6.88	6.42	6.06		7.33	6.81	6.40		7.85	7.31	6.84			7.76	7.24				7.59			7.85
18	7.62	6.98	6.51	6.14		7.45	6.91	6.50			7.43	6.95			7.85	7.37				7.73			
19	7.85	7.18	6.68	6.29		7.69	7.12	6.68			7.68	7.18				7.62				7.85			
20		7.38	6.86	6.45		7.85	7.33	6.87			7.85	7.40				7.85							
21		7.58	7.04	6.60			7.54	7.05				7.62											
22		7.79	7.21	6.76			7.75	7.24				7.84											
23		7.85	7.39	6.91			7.85	7.42				7.85											
24			7.56	7.07				7.61															
25			7.74	7.23				7.79															
26			7.85	7.38				7.85															
27				7.54																			
28				7.69																			
29				7.85																			
30																							
31																							
32																							
33																							
34																							

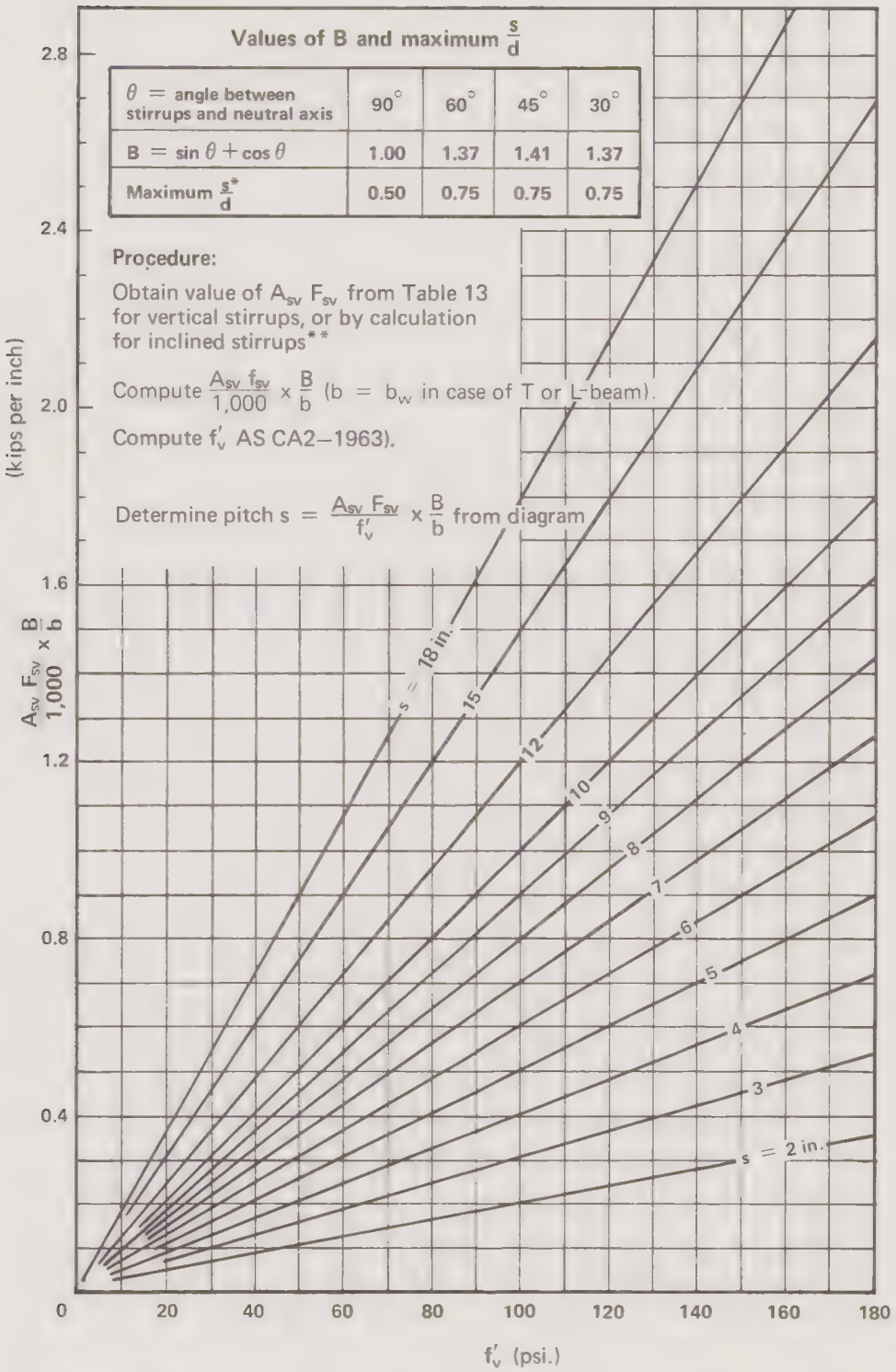
continued

Values of $\frac{A_{sv}\ F_{sv}}{1{,}000}$ for stirrup bar size #5

Effective depth d (in.)	F'_c = 2,000				F'_c = 2,500				F'_c = 3,000				F'_c = 3,500				F'_c = 4,000				F'_c = 5,000		
	F_s = Tensile stress in main bars (ksi.)																						
	15	20	25	30	15	20	25	30	15	20	25	30	15	20	25	30	15	20	25	30	20	25	30
9	8.04	7.64	**7.36**	**7.36**	8.25	7.80	7.46	**7.36**	8.53	8.03	7.64	**7.36**	8.78	8.22	7.79	7.46	8.98	8.38	7.91	7.54	8.63	8.16	7.78
9.5	8.19	7.77	7.46	**7.36**	8.43	7.94	7.59	**7.36**	8.73	8.20	7.79	7.48	9.01	8.42	7.97	7.61	9.24	8.60	8.11	7.72	8.86	8.37	7.97
10	8.34	7.89	7.57	**7.36**	8.60	8.09	7.72	7.43	8.94	8.38	7.95	7.62	9.24	8.62	8.14	7.77	9.50	8.83	8.31	7.89	9.10	8.58	8.16
10.5	8.49	8.02	7.68	7.41	8.78	8.24	7.85	7.54	9.14	8.55	8.10	7.75	9.47	8.82	8.32	7.93	9.75	9.05	8.51	8.07	9.34	8.79	8.35
11	8.64	8.15	7.79	7.50	8.95	8.39	7.98	7.66	9.34	8.73	8.26	7.89	9.70	9.02	8.50	8.09	10.0	9.27	8.70	8.25	9.57	9.00	8.54
11.5	8.79	8.27	7.90	7.60	9.13	8.54	8.11	7.78	9.55	8.91	8.41	8.03	9.94	9.22	8.67	8.25	10.3	9.50	8.90	8.42	9.81	9.21	8.73
12	8.93	8.40	8.01	7.70	9.30	8.69	8.24	7.89	9.75	9.08	8.56	8.17	10.2	9.42	8.85	8.40	10.5	9.72	9.10	8.60	10.0	9.43	8.92
12.5	9.08	8.53	8.12	7.80	9.48	8.84	8.37	8.01	9.96	9.26	8.72	8.31	10.4	9.62	9.03	8.56	10.8	9.94	9.30	8.78	10.3	9.64	9.11
13	9.23	8.65	8.23	7.89	9.65	8.99	8.50	8.12	10.2	9.43	8.87	8.44	10.6	9.82	9.20	8.72	11.0	10.2	9.49	8.95	10.5	9.85	9.30
13.5	9.38	8.78	8.34	7.99	9.82	9.14	8.63	8.24	10.4	9.61	9.03	8.58	10.9	10.0	9.38	8.88	11.3	10.4	9.69	9.13	10.8	10.1	9.49
14	9.53	8.91	8.45	8.09	10.0	9.29	8.76	8.36	10.6	9.79	9.18	8.72	11.1	10.2	9.56	9.04	11.6	10.6	9.89	9.31	11.0	10.3	9.68
14.5	9.68	9.03	8.56	8.19	10.2	9.44	8.89	8.47	10.8	9.96	9.34	8.86	11.3	10.4	9.73	9.19	11.8	10.8	10.1	9.48	11.2	10.5	9.87
15	9.83	9.16	8.67	8.28	10.3	9.58	9.02	8.59	11.0	10.1	9.49	9.00	11.6	10.6	9.91	9.35	12.1	11.1	10.3	9.66	11.5	10.7	10.1
15.5	9.98	9.29	8.78	8.38	10.5	9.73	9.15	8.70	11.2	10.3	9.64	9.13	11.8	10.8	10.1	9.51	12.3	11.3	10.5	9.84	11.7	10.9	10.2
16	10.1	9.41	8.89	8.48	10.7	9.88	9.28	8.82	11.4	10.5	9.80	9.27	12.0	11.0	10.3	9.67		11.5	10.7	10.0	11.9	11.1	10.4
16.5	10.3	9.54	9.00	8.57	10.9	10.0	9.41	8.94	11.6	10.7	9.95	9.41	12.3	11.2	10.4	9.83		11.7	10.9	10.2	12.2	11.3	10.6
17	10.4	9.67	9.11	8.67	11.0	10.2	9.54	9.05	11.8	10.8	10.1	9.55		11.4	10.6	9.99		12.0	11.1	10.4	12.3	11.5	10.8
17.5	10.6	9.79	9.22	8.77	11.2	10.3	9.68	9.17	12.0	11.0	10.3	9.68		11.6	10.8	10.1		12.2	11.3	10.5		11.7	11.0
18	10.7	9.92	9.33	8.87	11.4	10.5	9.81	9.28	12.2	11.2	10.4	9.82		11.8	11.0	10.3		12.3	11.5	10.7		12.0	11.2
19	11.0	10.2	9.55	9.06	11.7	10.8	10.1	9.52	12.3	11.5	10.7	10.1		12.2	11.3	10.6			11.7	11.1		12.3	11.6
20	11.3	10.4	9.77	9.26	12.1	11.1	10.3	9.75		11.9	11.0	10.4		12.3	11.7	10.9			12.3	11.4			12.0
21	11.6	10.7	9.99	9.45	12.3	11.4	10.6	9.98		12.2	11.3	10.6			12.0	11.3				11.8			12.3
22	11.9	10.9	10.2	9.65		11.7	10.8	10.2		12.3	11.7	10.9			12.3	11.6				12.1			
23	12.2	11.2	10.4	9.84		12.0	11.1	10.4			12.0	11.2				11.9				12.3			
24	12.3	11.4	10.7	10.0		12.3	11.4	10.7			12.3	11.5				12.2							
25		11.7	10.9	10.2			11.6	10.9				11.8				12.3							
26		11.9	11.1	10.4			11.9	11.1				12.0											
27		12.2	11.3	10.6			12.2	11.4				12.3											
28		12.3	11.5	10.8			12.3	11.6															
29			11.8	11.0				11.8															
30			12.0	11.2				12.1															
31			12.2	11.4				12.3															
32			12.3	11.6																			
33				11.8																			
34				12.0																			
36				12.3																			
38																							
40																							
42																							
44																							

Table values in bold type indicate that the embedded length l_a is negative. For these cases it is assumed that F_{sv} = 12,000 psi., so that $\frac{A_{sv}\ F_{sv}}{1{,}000}$ = 7.36 kips.

Diagram 14
Spacing (s) of Stirrups



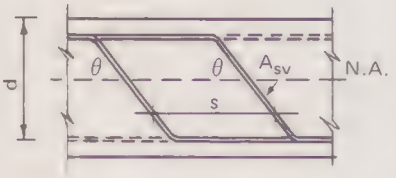
Examples 10 and 11 refer

* $\frac{s}{d} \leqslant \frac{1}{2} + \frac{1}{2} \cot \theta$

$\leqslant 0.75$ (AS CA2–1963).

** From AS CA2–1963, the tensile stress developed in the leg of a stirrup by a standard hook which embraces the longitudinal reinforcement may be taken as $6F'_c$ or 12,000 psi., whichever is the lesser, only when the stirrup is perpendicular to the longitudinal reinforcement. For other types of hooks or bends, or inclined stirrups, the anchorage value of the hook or bend and the value of A_{sv} F_{sv} for the stirrup should be computed in accordance with the requirements of the code.

Table 15
Values of $\frac{V'}{1{,}000jd}$ for Inclined Shear Reinforcement



Values of $\frac{V'}{1{,}000jd} = \frac{A_{sv}\, F_{sv}\, (\sin\theta + \cos\theta)}{1{,}000\, s}$ (kips per in.)

for single inclined bars with full end anchorage assumed.

Maximum $\frac{s}{d} = \frac{3}{8}\,(1 + \cot\theta) \leqslant \frac{3}{4}$

Maximum $f'_v = \frac{V'}{bjd} = 0.04\, F'_c + 40$, or $= 180$ psi.

Procedure: Enter table with computed value of $\frac{V'}{1{,}000jd}$; select diameter and spacing for inclined bars.

F_{sv}	Size No. for bars	Horizontal spacing of single inclined bars = s (in.)											
		2	3	4	6	8	10	12	15	18	21	24	27
		Angle of inclination $\theta = 30°$											
	2	.50	.34	.25	.17	.13	.10	.08	.07	.06	.05	.04	.04
	3	1.13	.75	.57	.38	.28	.23	.19	.15	.13	.11	.09	.08
	4	2.01	1.34	1.01	.67	.50	.40	.34	.27	.22	.19	.17	.15
	5	3.14	2.10	1.57	1.05	.79	.63	.52	.42	.35	.30	.26	.23
	6	4.53	3.02	2.26	1.51	1.13	.91	.75	.60	.50	.43	.38	.34
	7	6.16	4.11	3.08	2.05	1.54	1.23	1.03	.82	.68	.59	.51	.46
	8	8.05	5.36	4.02	2.68	2.01	1.61	1.34	1.07	.89	.77	.67	.60
		$\theta = 45°$											
	2	.52	.35	.26	.17	.13	.10	.09	.07	.06	.05	.04	.04
	3	1.17	.78	.59	.39	.29	.23	.20	.16	.13	.11	.10	.09
	4	2.08	1.39	1.04	.69	.52	.42	.35	.28	.23	.20	.17	.15
15,000	5	3.25	2.17	1.63	1.08	.81	.65	.54	.43	.36	.31	.27	.24
	6	4.69	3.12	2.34	1.56	1.17	.94	.78	.62	.52	.45	.39	.35
	7	6.38	4.25	3.19	2.13	1.59	1.28	1.06	.85	.71	.61	.53	.47
	8	8.33	5.55	4.17	2.78	2.08	1.67	1.39	1.11	.93	.79	.69	.62
		$\theta = 60°$											
	2	.50	.34	.25	.17	.13	.10	.08	.07	.06	.05	.04	.04
	3	1.13	.75	.57	.38	.28	.23	.19	.15	.13	.11	.09	.08
	4	2.01	1.34	1.01	.67	.50	.40	.34	.27	.22	.19	.17	.15
	5	3.14	2.10	1.57	1.05	.79	.63	.52	.42	.35	.30	.26	.23
	6	4.53	3.02	2.26	1.51	1.13	.91	.75	.60	.50	.43	.38	.34
	7	6.16	4.11	3.08	2.05	1.54	1.23	1.03	.82	.68	.59	.51	.46
	8	8.05	5.36	4.02	2.68	2.01	1.61	1.34	1.07	.89	.77	.67	.60
		$\theta = 30°$											
	2	.67	.45	.34	.22	.17	.13	.11	.09	.07	.06	.06	.05
	3	1.51	1.01	.75	.50	.38	.30	.25	.20	.17	.14	.13	.11
	4	2.68	1.79	1.34	.89	.67	.54	.45	.36	.30	.26	.22	.20
	5	4.19	2.79	2.10	1.40	1.05	.84	.70	.56	.47	.40	.35	.31
	6	6.03	4.02	3.02	2.01	1.51	1.21	1.01	.80	.67	.57	.50	.45
	7	8.21	5.48	4.11	2.74	2.05	1.64	1.37	1.10	.91	.78	.68	.61
	8	10.73	7.15	5.36	3.58	2.68	2.15	1.79	1.43	1.19	1.02	.89	.79
		$\theta = 45°$											
	2	.69	.46	.35	.23	.17	.14	.12	.09	.08	.07	.06	.05
	3	1.56	1.04	.78	.52	.39	.31	.26	.21	.17	.15	.13	.12
	4	2.78	1.85	1.39	.93	.69	.56	.46	.37	.31	.26	.23	.21
20,000	5	4.34	2.89	2.17	1.45	1.08	.87	.72	.58	.48	.41	.36	.32
	6	6.25	4.17	3.12	2.08	1.56	1.25	1.04	.83	.69	.60	.52	.46
	7	8.50	5.67	4.25	2.83	2.13	1.70	1.42	1.13	.94	.81	.71	.63
	8	11.11	7.40	5.55	3.70	2.78	2.22	1.85	1.48	1.23	1.06	.93	.82
		$\theta = 60°$											
	2	.67	.45	.34	.22	.17	.13	.11	.09	.07	.06	.06	.05
	3	1.51	1.01	.75	.50	.38	.30	.25	.20	.17	.14	.13	.11
	4	2.68	1.79	1.34	.89	.67	.54	.45	.36	.30	.26	.22	.20
	5	4.19	2.79	2.10	1.40	1.05	.84	.70	.56	.47	.40	.35	.31
	6	6.03	4.02	3.02	2.01	1.51	1.21	1.01	.80	.67	.57	.50	.45
	7	8.21	5.48	4.11	2.74	2.05	1.64	1.37	1.10	.91	.78	.68	.61
	8	10.73	7.15	5.36	3.58	2.68	2.15	1.79	1.43	1.19	1.02	.89	.79

Example 11 refers

Design and Investigation of Compression Members

Concentrically loaded, $\frac{h}{D} \leqslant 15$

The load carrying capacity of short concentrically loaded tied compression members designed in accordance with Australian Standard CA2—1963 consists of the sum of two parts, namely

(i) the load supported by the concrete, and
(ii) the load supported by the longitudinal reinforcement.

The design procedure is demonstrated in Example 24, and the capacity of the concrete and reinforcement, for a range of column sizes, is given in Table 16. The loads on bars are tabulated in Table 16 in terms of the load on cross-sectional areas of reinforcement equal to 1% A_g and 4% A_g, and in Table 17, in terms of the load on combinations of various numbers and sizes of bars.

For short helically reinforced compression members, the concentric load capacity is taken as the sum of

(i) the load supported by the concrete core,
(ii) the load supported by the longitudinal reinforcement, and
(iii) the load supported by the helical reinforcement, provided that the sum of the loads on the concrete core and helical reinforcement does not exceed 0.50 F'_c A_g.

The design procedure is demonstrated in Example 25, and the design data for a range of column sizes are given in Table 19.

It should be noted that, while AS CA2—1963 specifies a minimum and maximum cross-sectional area of longitudinal reinforcement for tied or helically reinforced compression members of 1% A_g and 8% A_g respectively, the use of areas in excess of 4% A_g requires that special attention be paid to possible difficulties in placing and compacting concrete. The maximum area of reinforcement that can be used will also be limited, of course, by bar spacing requirements as shown in Table 18 for helically reinforced members.

For long concentrically loaded compression members, for which $\frac{h}{D} > 15$, the permissible loads evaluated in accordance with the above procedures for short members must be reduced by multiplying by the factor $(1.6 - 0.04 \frac{h}{D})$.

Eccentrically loaded, $\frac{h}{D} \leqslant 15$

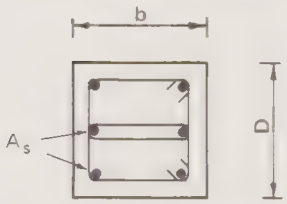
Eccentrically loaded symmetrical compression members may be designed or investigated with the aid of Diagrams 21 to 23. These diagrams, in accordance with the requirements of AS CA2—1963, are based on simple elastic theory, using n as the modular ratio. To avoid difficulty in preparing the diagrams, however, the transformed area of all reinforcement was assumed to be np, whereas the correct value for compressive reinforcement is $(n - 1)p$; accuracy is not significantly impaired by this substitution.

For design the column dimensions are first assumed, or are otherwise known, and the parameters $\frac{1{,}000\ N}{f_c b D}$ and $\frac{1{,}000\ Ne'}{f_c b D^2}$ are computed, as shown in Examples 26 and 27. The appropriate diagram is then entered to obtain np, from which A_s may be computed. The value of k is also taken from the diagram, for use in checking the maximum tensile stress in the reinforcement; when k is less than the limiting value given in Table 20, the tensile stress exceeds the permissible value and the section must be redesigned. Similarly it may also be necessary to check the maximum compressive stress in the reinforcement, as demonstrated.

Investigation is similar to design except that with np and $\frac{e'}{D}$ known, the diagrams are entered to determine $\frac{1{,}000\ N}{f_c b D}$. The stresses in concrete and reinforcement can then be computed as shown in Examples 28 and 29.

Example 24
Square Tied Column, Concentrically Loaded ($\frac{h}{b \text{ or } D} \leq 15$)

For a tied column subject to a concentric load P, determine:
(i) a suitable column size b x D, and
(ii) a suitable arrangement of ties and longitudinal reinforcement.



Given:
F'_c = 3,500 psi.
F_{sc} = 20,000 psi.
P = 400 kips.

A range of column sizes, with varying percentages of longitudinal reinforcement, could be selected so that the load carrying capacity of the concrete plus the reinforcement is equal to P. It is assumed here that a section requiring about 4% reinforcement will be satisfactory, provided of course that this amount of reinforcement can be fitted within the column cross-section.*

(1) From Table 16 (a) and (b), for F_{sc}/F'_c = 20,000/3,500 and A_s = 4% A_g, a 15 x 15 in. column has adequate load carrying capacity in that :

permissible load on concrete = 236 kips, and
permissible load on bars = 180 kips,
so that total column capacity = 416 kips.

Therefore try a 15 x 15 in. column.

(2) Compute actual load on bars = P — permissible load on concrete
= 400 — 236
= 164 kips.

From Table 17, for F_{sc} = 20,000 psi.: Select 6 # 11 bars, which will support 178 kips.

(3) Check actual A_s% $= \frac{178}{180} \times 4$

$= 3.95\% A_g$.**

Check spacing of bars within the cross-section. To maintain symmetry, three bars are located at each of two opposite faces of the section as shown above. In accordance with AS CA2—1963, and assuming lapped splices, ϕ3 ties and $1\frac{1}{2}$ in. clear cover, compute least column dimension to maintain minimum bar spacings :

$$\text{min. D} = 2 \times 1\tfrac{1}{2} + 2 \times \tfrac{3}{8} + 1\tfrac{3}{8} + 2 \times 3\tfrac{1}{2} \times 1\tfrac{3}{8}$$
$$= 3 + \tfrac{3}{4} + 1\tfrac{3}{8} + 9\tfrac{5}{8}$$
$$= 14\tfrac{3}{4} \text{ in.} < 15 \text{ in.}$$

Check maximum tie spacing.
From AS CA2—1963, maximum permissible tie spacing is the lesser of
s = 16 x longitudinal bar size
= 16 x $1\frac{3}{8}$ = 22 in.,
or s = least column dimension
= 15 in.

Therefore use 15 x 15 in. column section with 6 # 11 longitudinal bars and ϕ3 ties at 15 in. centres.

---

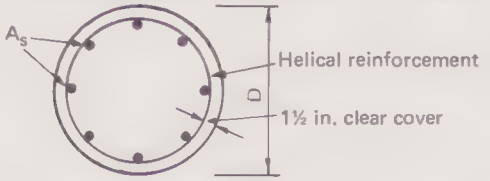
* It is not implied that columns containing about 4% reinforcement are necessarily the most satisfactory in any particular application.

** Where A_s exceeds 0.04 Ag the designer must check that the amount of reinforcement will not interfere with placing and compaction of concrete, particularly at splices and at junctions of members (see AS CA2—1963).

Example 25
Helically Reinforced Round Column, Concentrically Loaded ($\frac{h}{D} \leq 15$)

For a helically reinforced round column subject to a concentric load P, determine:

(i) a suitable column diameter D, and
(ii) a suitable arrangement of helical and longitudinal reinforcement.



Given:
F'_c = 3,000 psi.
F_{sc} = 16,500 psi.
P = 400 kips

Since a helically reinforced column can support a load which is the sum of the individual capacities of the concrete, the longitudinal reinforcement and the helical reinforcement, a number of different column sections may be designed to carry the same applied load P. The approach adopted here is to use Table 19 to estimate a column size which, with A_{sc} = 4% A_g (approx.) and an assumed helix size, will approximately support the applied load.* Once the column diameter is established, the amounts of longitudinal and helical reinforcement are adjusted if necessary to balance P more precisely.

(1) From Table 19 (a), (b) and (d), for F_{sc}/F'_c = 16,500/3,000 and assumed D = 18 in., ϕ3 helix at 2½ in. pitch, and A_s = 4% A_g:

permissible load on concrete = 159 kips,
permissible load on helical reinforcement = 55 kips, and
permissible load on longitudinal reinforcement = 168 kips.
Total column capacity = 382 kips.

Accept D = 18 in., and vary reinforcement to give P = 400 kips.

(2) Compute load to be supported by reinforcement = P − load on concrete
= 400 − 159
= 241 kips.

From Table 17, for F_{sc} = 16,500 psi. :
load on 9 # 10 bars, say = 182 kips.
Then load to be supported by helix = 241 − 182
= 59 kips.

From Table 19 (b): Select ϕ3 helix at 2¼ in. pitch, which can support 61 kips.

(3) Check actual $A_s\%$ = $\frac{182}{168}$ x 4
= 4.3%.**

From Table 19 (c), for F'_c = 3,000 psi. and D = 18 in. :
permissible load on concrete + helix = 382 kips.
Actual maximum load = 159 + 61
= 220 kips.

Check maximum number of bars in 18 in. diameter column.

From Table 18: for circumferential splice, maximum = 9 # 10;
for radial splice, maximum = 10 # 10.

Therefore use 18 in. diameter column, with 9 # 10 longitudinal bars and ϕ3 helix at 2¼ in. pitch.

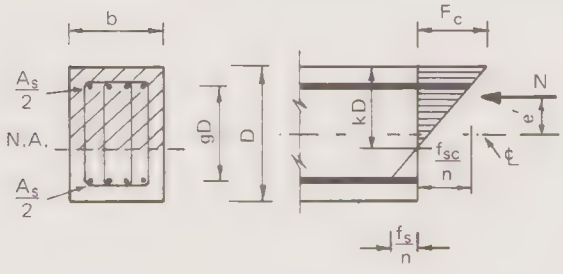
---

* It is not implied that columns containing about 4% reinforcement are necessarily the most satisfactory in any particular application.

** Since A_{sc} exceeds 0.04 A_g, the designer is required to check that the amount of reinforcement will not interfere with placing and compaction of concrete, particularly at splices and at junctions of members (see AS CA2–1963). This check is omitted here.

Example 26
Rectangular Tied Column, Eccentrically Loaded ($\frac{h}{b \text{ or } D} \leq 15$) – Design

For a rectangular section of specified dimensions and subject to an axial load N combined with a bending moment M, determine suitable longitudinal reinforcement A_s.



Given:

b	=	18 in.
D	=	24 in.
g	=	0.80.
F_c'	=	3,000 psi.
F_c	=	1,350 psi.
F_{sc}	=	16,500 psi.
F_s	=	20,000 psi.
n	=	12.
N	=	220 kips.
M	=	2,200 in. kips.

(1) Compute

$$e' = \frac{M}{N}$$

$$= \frac{2{,}200}{220}$$

$$= 10 \text{ in.},$$

$$\text{and } \frac{e'}{D} = \frac{10}{24}$$

$$= 0.42.$$

(2) Compute

$$\frac{1{,}000\ N}{F_c\ b\ D} = \frac{1{,}000 \times 220}{1{,}350 \times 18 \times 24}$$

$$= 0.377,$$

$$\text{and } \frac{1{,}000\ Ne'}{F_c\ b\ D^2} = 0.377 \times 0.42$$

$$= 0.158.$$

From Diagram 21, for g = 0.8 : np = 0.27 and k = 0.64.

Then

$$A_s = \frac{np}{n} Ag$$

$$= \frac{0.27}{12} \times 18 \times 24$$

$$= 9.7 \text{ sq. in.}$$

Check

$$p = \frac{np}{n}$$

$$= 0.0225 > 0.01 \text{ and } < 0.08.$$

Select 8 # 10 bars, for which A_s = 9.82 sq. in. Place 4 bars adjacent to each 18 in. face of the column.

For bar spacing in accordance with AS CA2–1963, and assuming 1½ in. clear cover and ϕ3 ties, compute minimum width b to contain 4 # 10 bars:

$$\text{min. b} = 2 \times 1\tfrac{1}{2} + 2 \times \tfrac{3}{8} + 1\tfrac{1}{4} + 1\tfrac{1}{4}(3\tfrac{1}{2} + 2\tfrac{1}{2} + 3\tfrac{1}{2})$$

$$= 16\tfrac{7}{8} \text{ in.} < 18 \text{ in.}$$

(3) Check tensile stress in reinforcement near tension face.*

From Table 20, for F_c = 1,350 psi., F_s = 20,000 psi. and g = 0.8:

limiting k = 0.403.

Actual k = 0.64, which exceeds 0.403; therefore $f_s < 20{,}000$ psi.

* The tension face is on the opposite side of the centroidal axis to N and the compression face on the same side.

Example 26 (ctd.)

Check compressive stress in reinforcement near compression face.

Compute $nF_c = 12 \times 1{,}350$

$= 16{,}200$ psi.

Therefore $f_{sc} < 16{,}200 < F_{sc}$.*

(4) Check permissible concentric load P on section assuming 8 # 10 bars.

From Table 16, for $F'_c = 3{,}000$ psi. and b x D $\doteq$ 18 x 24 in :

load on concrete $= 389$ kips.

From Table 17, for $F_{sc} = 16{,}500$ psi. and 8 # 10 bars :

load on bars $= 162$ kips.

$P = 551$ kips $> N$.

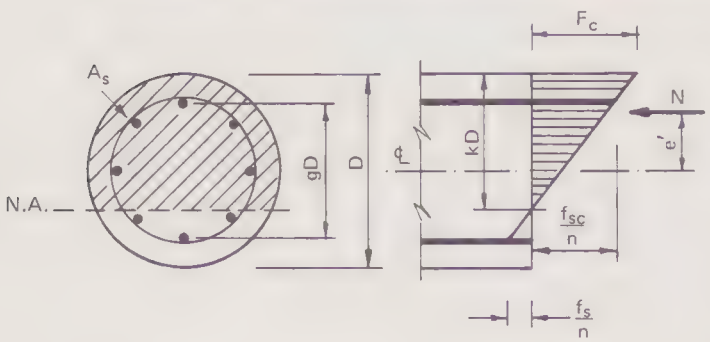
Therefore 8 # 10 bars are satisfactory in this column section.

---

* This check follows from Equation (61), viz. $f_{sc} = n f_c (1 - \frac{1-g}{2k})$, with $f_c = F_c$. When the section is eccentrically loaded, the expression within the brackets is always less than 1, so that $f_{sc} < nF_c$. When $f_{sc} > nF_c$, f_{sc} should be calculated from Equation (61), and if f_{sc} exceeds F_{sc}, the section redesigned using a reduced value of f_c or by adjusting the column dimensions.

Example 27
Helically Reinforced Round Column, Eccentrically Loaded ($\frac{h}{D} \leqslant 15$) — Design

For a circular section of specified diameter and subject to an axial load N combined with a bending moment M, determine suitable longitudinal reinforcement A_s.



Given:

D = 21 in.
g = 0.76.
F'_c = 3,000 psi.
F_c = 1,350 psi.
F_{sc} = 16,500 psi.
F_{st} = 20,000 psi.
n = 12.
N = 260 kips.
M = 1,100 in. kips.

(1) Compute $e' = \frac{M}{N}$

$= \frac{1{,}100}{260}$

$= 4.2$ in.,

and $\frac{e'}{D} = \frac{4.2}{21}$

$= 0.20$.

(2) Compute

$$\frac{1{,}000\ N}{F_c\ D^2} = \frac{1{,}000 \times 260}{1{,}350 \times 21^2}$$

$$= 0.437,$$

$$\text{and } \frac{1{,}000\ Ne'}{F_c\ D^3} = 0.437 \times 0.20$$

$$= 0.087.$$

From Diagram 23, for $g = 0.7$: $np = 0.48$ and $k = 0.78$;
for $g = 0.8$: $np = 0.40$ and $k = 0.82$.
By interpolation, for $g = 0.76$: $np = 0.45$, $k = 0.80$.

Then

$$A_s = \frac{np}{n} Ag$$

$$= \frac{0.45}{12} \times \frac{\pi \times 21^2}{4}$$

$$= 13.0 \text{ sq. in.}$$

Check

$$p = \frac{np}{n}$$

$$= 0.0375 > 0.01 \text{ and } < 0.08$$

Select 11 # 10 bars $A_s = 13.5$ sq. in.

Check maximum number of bars in 21 in. dia. column.
From Table 18 : for circumferential splice, maximum = 11 # 10,
and for radial splice, maximum = 13 # 10.

(3) Check tensile stress in reinforcement near tensile face.*

From Table 20, for $F_c = 1{,}350$ psi., $F_s = 20{,}000$ psi. and $g = 0.76$:

limiting $k = 0.39$.

Actual $k = 0.80$, which exceeds 0.39; therefore $f_s < 20{,}000$ psi.

Check compressive stress in reinforcement near compression face.

Compute $nF_c = 12 \times 1{,}350$
$= 16{,}200$ psi.

Therefore $f_{sc} < 16{,}200 < F_{sc}$.**

(4) Check permissible concentric load P on section.

From Tables 17 and 19, for $F_{sc}/F'_c = 16{,}500/3{,}000$, D = 21 in. and $\phi 3$ helical reinforcement at 3 in. pitch (assumed):

Load on concrete = 229 kips,
Load on longitudinal bars (11 # 10) = 223 kips, and
Load on helical reinforcement = 55 kips.
$P = 507$ kips $> N$.

Therefore 11 # 10 bars are satisfactory in this column section.

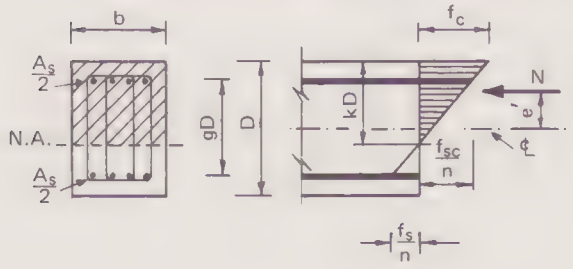
---

* The tension face is on the opposite side of the centroidal axis to N, and the compression face on the same side.

** See footnote to Example 26 opposite.

Example 28
Rectangular Tied Column, Eccentrically Loaded ($\frac{h}{b \text{ or } D} \leq 15$) – Investigation

For a given rectangular section symmetrically reinforced as shown and subject to an axial load N combined with a bending moment M, determine f_c, f_s and f_{sc}.



Given:

b = 18 in. D = 24 in.
g = 0.80.
A_s = 9.82 sq. in.
n = 12.
N = 220 kips.
M = 2,200 in. kips.

(1) Compute

$$e' = \frac{M}{N}$$
$$= \frac{2,200}{220}$$
$$= 10 \text{ in.},$$
$$\text{and } \frac{e'}{D} = \frac{10}{24}$$
$$= 0.42.$$

(2) Compute

$$np = \frac{n A_s}{b D}$$
$$= \frac{12 \times 9.82}{18 \times 24}$$
$$= 0.27.$$

From Diagram 21, for g = 0.8, $\frac{e'}{D}$ = 0.42 and np = 0.27:

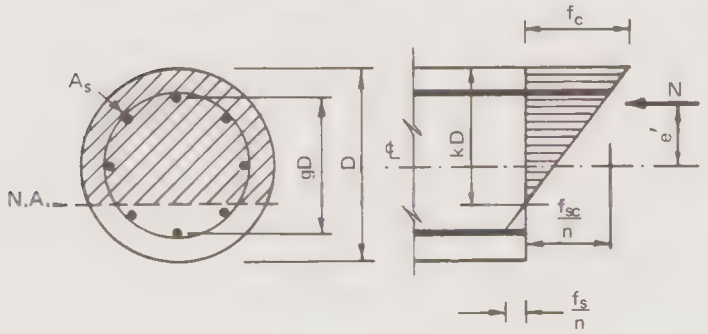
$$\frac{1,000 N}{f_c b D} = 0.38 \text{ and } k = 0.65.$$

(3) Compute

$$f_c = \frac{1,000 N}{0.38 b D}$$
$$= \frac{1,000 \times 220}{0.38 \times 18 \times 24}$$
$$= 1,340 \text{ psi.};$$
$$f_s = nf_c \left(\frac{1+g}{2k} - 1\right)$$
$$= 12 \times 1,340 \left(\frac{1+0.8}{2 \times 0.65} - 1\right)$$
$$= 6,200 \text{ psi. (tension)};$$
$$f_{sc} = nf_c \left(1 - \frac{1-g}{2k}\right)$$
$$= 12 \times 1,340 \left(1 - \frac{1-0.8}{2 \times 0.65}\right)$$
$$= 13,600 \text{ psi.}$$

Example 29
Helically Reinforced Round Column, Eccentrically Loaded ($\frac{h}{D} \leq 15$) – Investigation

For a given circular section subject to an axial load N combined with a bending moment M, determine f_c, f_s and f_{sc}.



Given:

D = 21 in.
g = 0.76.
A_s = 13.5 sq. in.
n = 12.
N = 260 kips.
M = 1,100 in. kips.

(1) Compute

$$e' = \frac{M}{N}$$
$$= \frac{1,100}{260}$$
$$= 4.2 \text{ in.},$$
$$\text{and } \frac{e'}{D} = \frac{4.2}{21}$$
$$= 0.20.$$

(2) Compute

$$np = \frac{nA_s}{\pi \frac{D^2}{4}}$$
$$= \frac{12 \times 13.5 \times 4}{\pi \times 21^2}$$
$$= 0.47.$$

From Diagram 23, for $\frac{e'}{D} = 0.20$, np = 0.47 and g = 0.7 :

$$\frac{1,000\ N}{f_c\ D^2} = 0.43 \text{ and } k = 0.78.$$

For g = 0.8 : $\frac{1,000\ N}{f_c\ D^2} = 0.46$ and k = 0.82.

By interpolation, for g = 0.76: $\frac{1,000\ N}{f_c\ D^2} = 0.45$ and k = 0.80.

(3) Compute

$$f_c = \frac{1,000\ N}{0.45\ D^2}$$
$$= \frac{1,000 \times 260}{0.45 \times 21^2}$$
$$= 1,310 \text{ psi};$$
$$f_s = nf_c\left(\frac{1+g}{2k} - 1\right)$$
$$= 12 \times 1,310\left(\frac{1+0.76}{2 \times 0.80} - 1\right)$$
$$= 1,600 \text{ psi. (tension)};$$
$$f_{sc} = nf_c\left(1 - \frac{1-g}{2k}\right)$$
$$= 12 \times 1,310\left(1 - \frac{1-0.76}{2 \times 0.80}\right)$$
$$= 12,400 \text{ psi.}$$

Table 16
Load on Gross Section for Short Concentrically Loaded Tied Compression Members ($\frac{h}{D} \leq 15$)

Load on Gross Section: $P = \frac{0.30\ F'_c A_g + F_{sc} A_s}{1{,}000}$ (kips)

Note: Min. A_s = 0.01 A_g (1%), or 4 ϕ 4 bars (0.79 sq. in.).
Max. A_s = 0.08 A_g (8%).
Min. ties = ϕ2 at a maximum spacing equal to the lesser of
(i) 16 x least diameter of longitudinal reinforcement, or
(ii) least dimension of column.

Column size (in. x in.)	Gross Area A_g (sq. in.)	(a) Load on Concrete F'_c				(b) Load on Bars * for A_s = 1% A_g and 4% A_g					
						F_{sc} = 16,500		F_{sc} = 20,000		F_{sc} = 23,000	
		3,000	3,500	4,000	5,000	A_s = 1%	A_s = 4%	A_s = 1%	A_s = 4%	A_s = 1%	A_s = 4%
6 x 6	36	32.4	37.8	43.2	54.0	**5.9**	23.8	**7.2**	28.8	**8.3**	33.1
7	42	37.8	44.1	50.4	63.0	**6.9**	27.7	**8.4**	33.6	**9.7**	38.6
8	48	43.2	50.4	57.6	72.0	**7.9**	31.7	**9.6**	38.4	**11.0**	44.2
9	54	48.6	56.7	64.8	81.0	**8.9**	35.6	**10.8**	43.2	**12.4**	49.7
8 x 8	64	57.6	67.2	76.8	96.0	**10.6**	42.2	**12.8**	51.2	**14.7**	58.9
9	72	64.8	75.6	86.4	108	**11.9**	47.5	**14.4**	57.6	**16.6**	66.2
10	80	72.0	84.0	96.0	120	13.2	52.8	16.0	64.0	18.4	73.6
12	96	86.4	101	115	144	15.8	63.4	19.2	76.8	22.1	88.3
14	112	101	118	134	168	18.5	73.9	22.4	89.6	25.8	103
9 x 9	81	72.9	85.1	97.2	122	13.4	53.5	16.2	64.8	18.6	74.5
12	108	97.2	113	130	162	17.8	71.3	21.6	86.4	24.8	99.4
15	135	122	142	162	203	22.3	89.1	27.0	108	31.1	124
18	162	146	170	194	243	26.7	107	32.4	130	37.3	149
12 x 12	144	130	151	173	216	23.8	95.0	28.8	115	33.1	132
15	180	162	189	216	270	29.7	119	36.0	144	41.4	166
18	216	194	227	259	324	35.6	143	43.2	173	49.7	199
21	252	227	265	302	378	41.6	166	50.4	202	58.0	232
24	288	259	302	346	432	47.5	190	57.6	230	66.2	265
15 x 15	225	203	236	270	338	37.1	149	45.0	180	51.8	207
18	270	243	284	324	405	44.6	178	54.0	216	62.1	248
21	315	284	331	378	473	52.0	208	63.0	252	72.5	290
24	360	324	378	432	540	59.4	238	72.0	288	82.8	331
27	405	365	425	486	608	66.8	267	81.0	324	93.2	373
18 x 18	324	292	340	389	486	53.5	214	64.8	259	74.5	298
21	378	340	397	454	567	62.4	249	75.6	302	86.9	348
24	432	389	454	518	648	71.3	285	86.4	346	99.4	397
27	486	437	510	583	729	80.2	321	97.2	389	112	447
30	540	486	567	648	810	89.1	356	108	432	124	497
21 x 21	441	397	463	529	662	72.8	291	88.2	353	101	406
24	504	454	529	605	756	83.2	333	101	403	116	464
27	567	510	595	680	851	93.6	374	113	454	130	522
30	630	567	662	756	945	104	416	126	504	145	580
36	756	680	794	907	1,134	125	499	151	605	174	696
24 x 24	576	518	605	691	864	95.0	380	115	461	132	530
27	648	583	680	778	972	107	428	130	518	149	596
30	720	648	756	864	1,080	119	475	144	576	166	662
36	864	778	907	1,037	1,296	143	570	173	691	199	795
42	1,008	907	1,058	1,210	1,512	166	665	202	806	232	927
—	1,000	900	1,050	1,200	1,500	165	660	200	800	230	920

Examples 24 and 26 refer

* The loads on bars shown above in bold type are less than the permissible loads on 4 ϕ 4 bars, the minimum A_s (see Table 17).

Table 17
Maximum Permissible Compression Loads on Bars

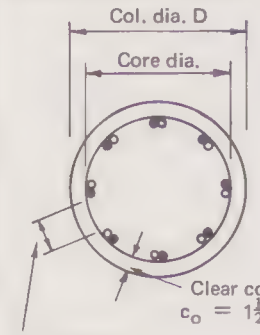
Load on bars = $\frac{F_{sc} A_s}{1{,}000}$ (kips)

Bar Size No.	Total number of bars										
	4	6	8	10	12	14	16	18	20	22	24
	F_{sc} = 16,500										
4	13.0	19.4	25.9	32.4	38.9	45.4	51.8	58.3	64.8	71.3	77.8
5	20.2	30.4	40.5	50.6	60.7	70.9	81.0	91.1	101	111	121
6	29.2	43.7	58.3	72.9	87.5	102	117	131	146	160	175
7	39.7	59.5	79.4	99.2	119	139	159	179	198	218	238
8	51.8	77.8	104	130	156	181	207	233	259	285	311
9	65.6	98.4	131	164	197	230	262	295	328	361	394
10	90.0	121	162	202	243	283	324	364	405	445	486
11	98.0	147	196	245	294	343	392	441	490	539	588
	F_{sc} = 20,000										
4	15.7	23.6	31.4	39.3	47.1	55.0	62.8	70.7	78.5	86.4	94.2
5	24.5	36.8	49.1	61.4	73.6	85.9	98.2	110	123	135	147
6	35.3	53.0	70.7	88.4	106	124	141	159	177	194	212
7	48.1	72.2	96.2	120	144	168	192	216	241	265	289
8	62.8	94.2	126	157	188	220	251	283	314	346	377
9	79.5	119	159	199	239	278	318	358	398	437	477
10	98.2	147	196	245	295	344	393	442	491	540	589
11	119	178	238	297	356	416	475	535	594	653	713
	F_{sc} = 23,000										
4	18.1	27.1	36.1	45.2	54.2	63.2	72.3	81.3	90.3	99.4	108
5	28.2	42.3	56.5	70.6	84.7	98.8	113	127	141	155	169
6	40.6	61.0	81.3	102	122	142	163	183	203	224	244
7	55.3	83.0	111	138	166	194	221	249	277	304	332
8	72.3	108	145	181	217	253	289	325	361	397	434
9	91.4	137	183	229	274	320	366	412	457	503	549
10	113	169	226	282	339	395	452	508	565	621	677
11	137	205	273	342	410	478	546	615	683	751	820

Examples 24, 25, 26 and 27 refer

Table 18
Recommended Maximum Number of Bars for Helically Reinforced Compression Members

Circumferential Splice



Number of bars*

$$= \frac{\pi(D - 2c_o - 2\phi_1 - \phi_2)}{1\frac{1}{2} + 2\phi_2}$$

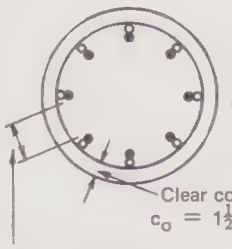
for $\phi_2 \leqslant 1$ in.,

or $$= \frac{\pi(D - 2c_o - 2\phi_1 - \phi_2)}{3\frac{1}{2}\phi_2}$$

for $\phi_2 \geqslant 1\frac{1}{8}$ in.

ϕ_1 = dia. of helical reinforcement.
ϕ_2 = dia. of main bars.

Radial Splice



Number of bars*

$$= \frac{\pi(D - 2c_o - 2\phi_1 - 3\phi_2)}{1\frac{1}{2} + \phi_2}$$

for $\phi_2 \leqslant 1$ in.,

or $$= \frac{\pi(D - 2c_o - 2\phi_1 - 3\phi_2)}{2\frac{1}{2}\phi_2}$$

for $\phi_2 \geqslant 1\frac{1}{8}$ in.

ϕ_1 = dia. of helical reinforcement.
ϕ_2 = dia. of main bars.

Col. dia. D (in.)	Core dia. (in.)	Main Bar Size Nº (ϕ_2)								Main Bar Size Nº (ϕ_2)								Assumed helix size Nº (ϕ_1)
		4	5	6	7	8	9	10	11	4	5	6	7	8	9	10	11	
12	9	9	8	7	7	6				10	9	8	7	6				
13	10	11	9	8	8	7	6			12	10	9	8	7	6			
14	11	12	11	9	9	8	7	6		13	12	11	10	9	7	6		3
15	12	13	12	11	10	9	8	7	6	15	13	12	11	10	8	7	6	
16	13	14	13	12	11	10	8	7	7	16	15	13	12	11	9	8	7	
17	14	15	14	12	11	10	9	8	7	18	16	15	13	12	10	9	8	
18	15	16	15	13	12	11	10	9	8	19	17	16	15	13	11	10	9	
20	17	19	17	15	14	13	11	10	9	22	20	19	17	16	14	12	10	4
21	18	20	18	17	15	14	12	11	10	24	22	20	19	17	15	13	11	
24	21	24	22	20	18	17	15	13	12	29	26	24	22	21	18	16	14	
27	24	27	25	23	21	19	17	15	13	33	30	28	26	24	21	19	17	
30	27	31	28	26	24	22	19	17	15	38	35	32	30	28	24	22	19	
33	30	35	32	29	26	24	22	19	17	42	39	37	34	32	28	25	22	5
36	33	39	35	32	29	27	24	21	19	47	44	41	38	36	31	28	25	
42	39	46	42	38	35	32	29	26	23	56	53	49	46	43	38	34	30	

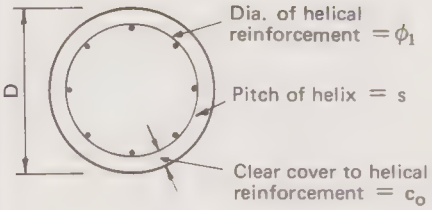
Examples 25 and 27 refer

* These equations assume that the spacing between adjacent bars is measured along the circumference of the pitch circle of bar centres. Table values are rounded to the next smaller whole number.

Table 19

Load on Gross Section for Short Concentrically Loaded Helically Reinforced Compression Members ($\frac{h}{D} \leqslant 15$)

Load on Gross Section : $P = \dfrac{0.30\,F'_c\,A_k + 27{,}000\,A_b + F_{sc}\,A_s}{1{,}000}$ (kips),



where $A_k = \dfrac{\pi(D - 2c_o)^2}{4}$

$A_b = \dfrac{\frac{\pi\phi_1^2}{4}\sqrt{[\pi\,(D - 2c_o - \phi_1)]^2 + s^2}}{s}$

A_s = total area of longitudinal bars

$A_g = \dfrac{\pi D^2}{4}$

Table based on clear cover $c_o = 1\frac{1}{2}$ in.

Note: (i) Min. $A_s = 0.01\,A_g$ (1%) or 6 ϕ 4 (1.18 sq. in.);

(ii) Max. $A_s = 0.08\,A_g$ (8%);

(iii) $\dfrac{0.30\,F'_c\,A_k + 27{,}000\,A_b}{1{,}000} \leqslant \dfrac{0.50\,F'_c\,A_g}{1{,}000}$

(a) Load on Concrete $= \dfrac{0.30\,F'_c\,A_k}{1{,}000}$ (kips)

F'_c	Column size D (in.) and A_k (sq. in.)														
	12	13	14	15	16	17	18	20	21	24	27	30	33	36	42
	63.6	78.5	95.0	113	133	154	177	227	254	346	452	573	707	855	1,195
3,000	13.7	70.7	85.5	102	120	139	159	204	229	312	407	515	636	770	1,075
3,500	66.8	82.5	99.8	119	139	162	186	238	267	364	475	601	742	898	1,254
4,000	76.3	94.2	114	136	159	185	212	272	305	416	543	687	848	1,026	1,434
5,000	95.4	118	143	170	199	231	265	341	382	520	679	859	1,060	1,283	1,792

(b) Load on Helical Reinforcement $= 27\,A_b$ (kips)

s (in.)	Helical reinforcement : ϕ2														
$1\frac{1}{2}$	24.3	27.1	29.9	32.6	35.4	38.2	41.0	46.5	49.3	57.6	65.9	74.3	82.6	90.9	108
$1\frac{3}{4}$			25.6	28.0	30.4	32.7	35.1	39.9	42.3	49.4	56.5	63.7	70.8	77.9	92.2
2				24.5	26.6	28.7	30.7	34.9	37.0	43.2	49.5	55.7	62.0	68.2	80.7
$2\frac{1}{4}$						25.5	27.3	31.0	32.9	38.4	44.0	49.5	55.1	60.6	71.7
$2\frac{1}{2}$							24.6	27.9	29.6	34.6	39.6	44.6	49.6	54.6	64.6
$2\frac{3}{4}$								25.4	26.9	31.5	36.0	40.5	45.1	49.6	58.7
3									24.7	28.8	33.0	37.2	41.3	45.5	53.8
	Helical reinforcement : ϕ3														
$1\frac{1}{2}$	54.0	60.2	66.4	72.7	78.9	85.2	91.4	104	110	129	148	166	185	204	241
$1\frac{3}{4}$			57.0	62.3	67.7	73.0	78.4	89.1	94.4	110	127	143	159	175	207
2				54.5	59.2	63.9	68.6	77.9	82.6	96.7	111	125	139	153	181
$2\frac{1}{4}$						56.8	61.0	69.3	73.5	85.9	98.4	111	123	136	161
$2\frac{1}{2}$							54.9	62.4	66.1	77.4	88.6	99.8	111	122	145
$2\frac{3}{4}$								56.7	60.1	70.3	80.5	90.8	101	111	132
3									55.1	64.5	73.8	83.2	92.6	102	121
	Helical reinforcement : ϕ4														
$1\frac{1}{2}$	94.5	106	117	128	139	150	161	183	194	228	261	294	328	361	428
$1\frac{3}{4}$			100	110	119	129	138	157	167	195	224	252	281	309	366
2				95.9	104	113	121	138	146	171	196	221	246	271	321
$2\frac{1}{4}$						100	107	122	130	152	174	196	218	241	285
$2\frac{1}{2}$							96.7	110	117	137	157	177	197	217	257
$2\frac{3}{4}$								100	106	124	142	161	179	197	233
3									97.3	114	131	147	164	181	214
	Helical reinforcement : ϕ5														
2				148	161	174	187	213	226	265	304	343	382	421	499
$2\frac{1}{4}$						155	166	190	201	236	270	305	340	375	444
$2\frac{1}{2}$							150	171	181	212	243	275	306	337	400
$2\frac{3}{4}$								155	165	193	221	250	278	306	363
3									151	177	203	229	255	281	333

continued

(c) Maximum Load on Concrete + Helical Reinforcement = $\frac{0.50\ F'_c\ A_g}{1{,}000}$ (kips).

F'_c	Column size D (in.) and A_g (sq. in.)														
	12	13	14	15	16	17	18	20	21	24	27	30	33	36	42
	113	**133**	**154**	**177**	**201**	**227**	**254**	**314**	**346**	**452**	**573**	**707**	**855**	**1,018**	**1,385**
3,000	170	199	231	265	302	341	382	471	520	679	859	1,060	1,283	1,527	2,078
3,500	198	232	269	309	352	397	445	550	606	792	1,002	1,237	1,497	1,781	2,425
4,000	226	266	308	353	402	454	509	628	693	905	1,145	1,414	1,711	2,036	2,771
5,000	283	332	385	442	503	568	636	785	866	1,131	1,431	1,767	2,138	2,545	3,464

(d) Load on Longitudinal Bars** = $F_{sc}\ A_s$ (kips).

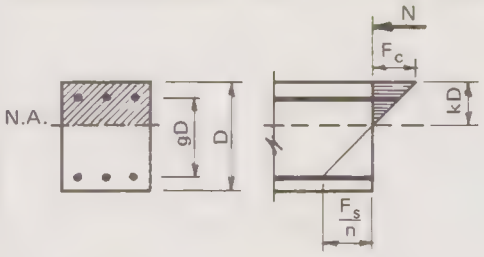
A_s/A_g %															
	F_{sc} = 16,500 psi.														
1%	**18.7**	21.9	25.4	29.2	33.2	37.5	42.0	51.8	57.2	74.6	94.5	117	141	168	229
4%	74.6	87.6	102	117	133	150	168	207	229	299	378	467	565	672	914
	F_{sc} = 20,000 psi.														
1%	**22.6**	26.6	30.8	35.3	40.2	45.4	50.9	62.8	69.3	90.5	115	141	171	204	277
4%	90.5	106	123	141	161	182	204	251	277	362	458	565	684	814	1,108
	F_{sc} = 23,000 psi.														
1%	**26.0**	30.5	35.4	40.6	46.2	52.2	58.5	72.3	79.7	104	132	163	197	234	319
4%	104	122	142	163	185	209	234	289	319	416	527	650	787	936	1,275

Examples 25 and 27 refer

* Parts (a), (b) and (d) of Table 19 are applicable to both round and square helically reinforced compression members of diameter or side dimension D. Tabulated values in Part (c) apply to round columns only.

** The loads on bars shown above in bold type are less than the permissible loads on 6 ϕ4 bars, the minimum A_s (see Table 17).

Table 20
Coefficient (k) for Limiting Tensile Stresses in Reinforcement in Eccentrically Loaded Symmetrical Compression Members



Limiting value of k $= \dfrac{1+g}{2(\frac{F_s}{nF_c}+1)}$

Procedure:

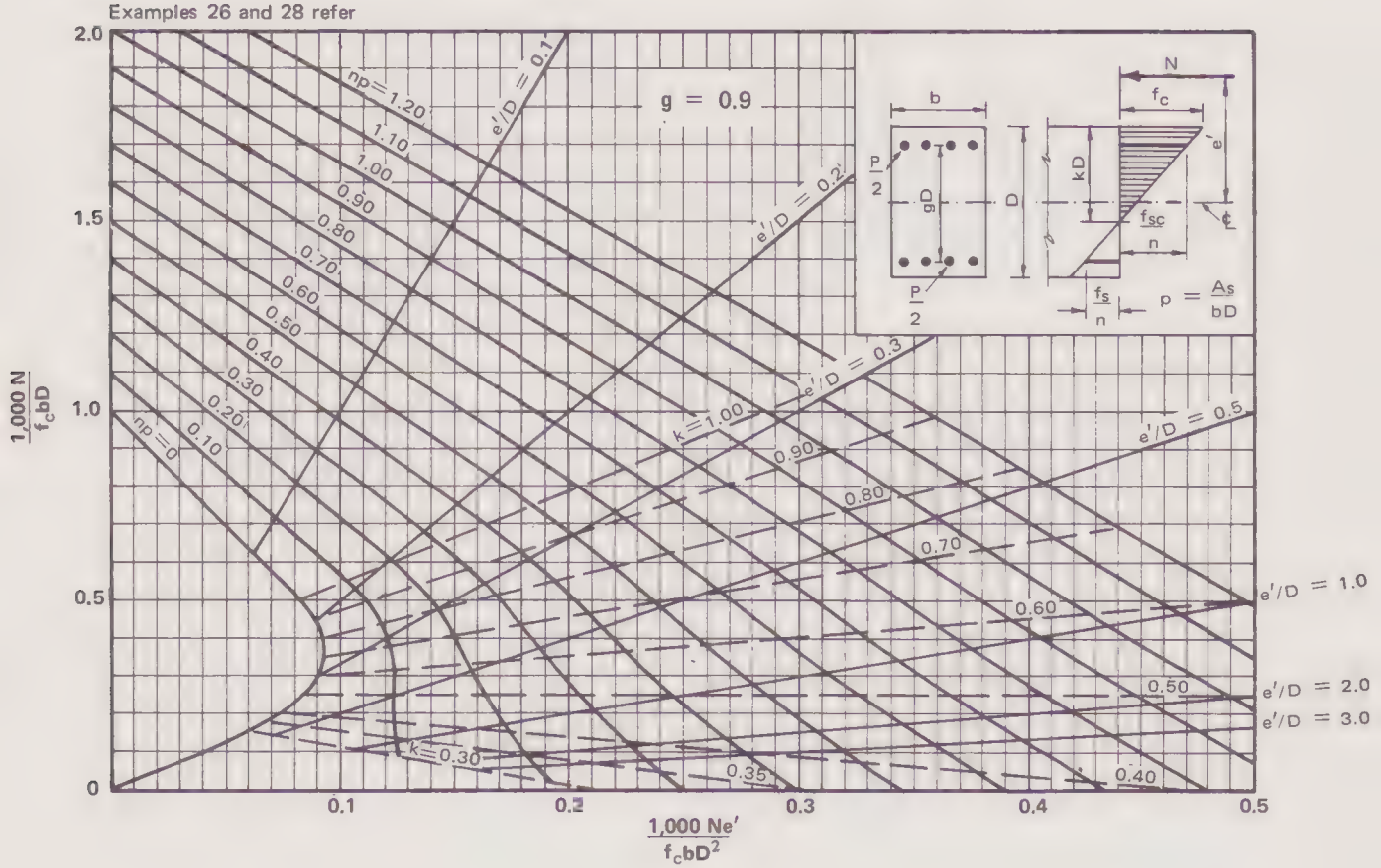
Enter table with known values of F_c, n, F_s and g. Select limiting value of k below which tensile stress in reinforcement exceeds F_s.

$\frac{F'_c}{F_c}$ n	g	F_s				
		15,000	18,000	20,000	25,000	30,000
3,000	0.9	.493	.450	.425	.374	.333
1,350	0.8	.467	.426	.403	.354	.316
12	0.7	.441	.403	.380	.334	.298
	0.6	.415	.379	.358	.315	.281
3,500	0.9	.509	.466	.441	.389	.348
1,575	0.8	.482	.441	.418	.368	.329
11	0.7	.456	.417	.395	.348	.311
	0.6	.429	.392	.371	.327	.293
4,000	0.9	.518	.475	.450	.398	.356
1,800	0.8	.491	.450	.426	.377	.338
10	0.7	.464	.425	.403	.356	.319
	0.6	.436	.400	.379	.335	.300
5,000	0.9	.546	.503	.478	.425	.383
2,250	0.8	.517	.476	.453	.403	.363
9	0.7	.488	.450	.428	.380	.343
	0.6	.460	.424	.402	.358	.322

Examples 26 and 27 refer

Diagram 21
Eccentrically Loaded Rectangular Compression Members with Ties

Examples 26 and 28 refer



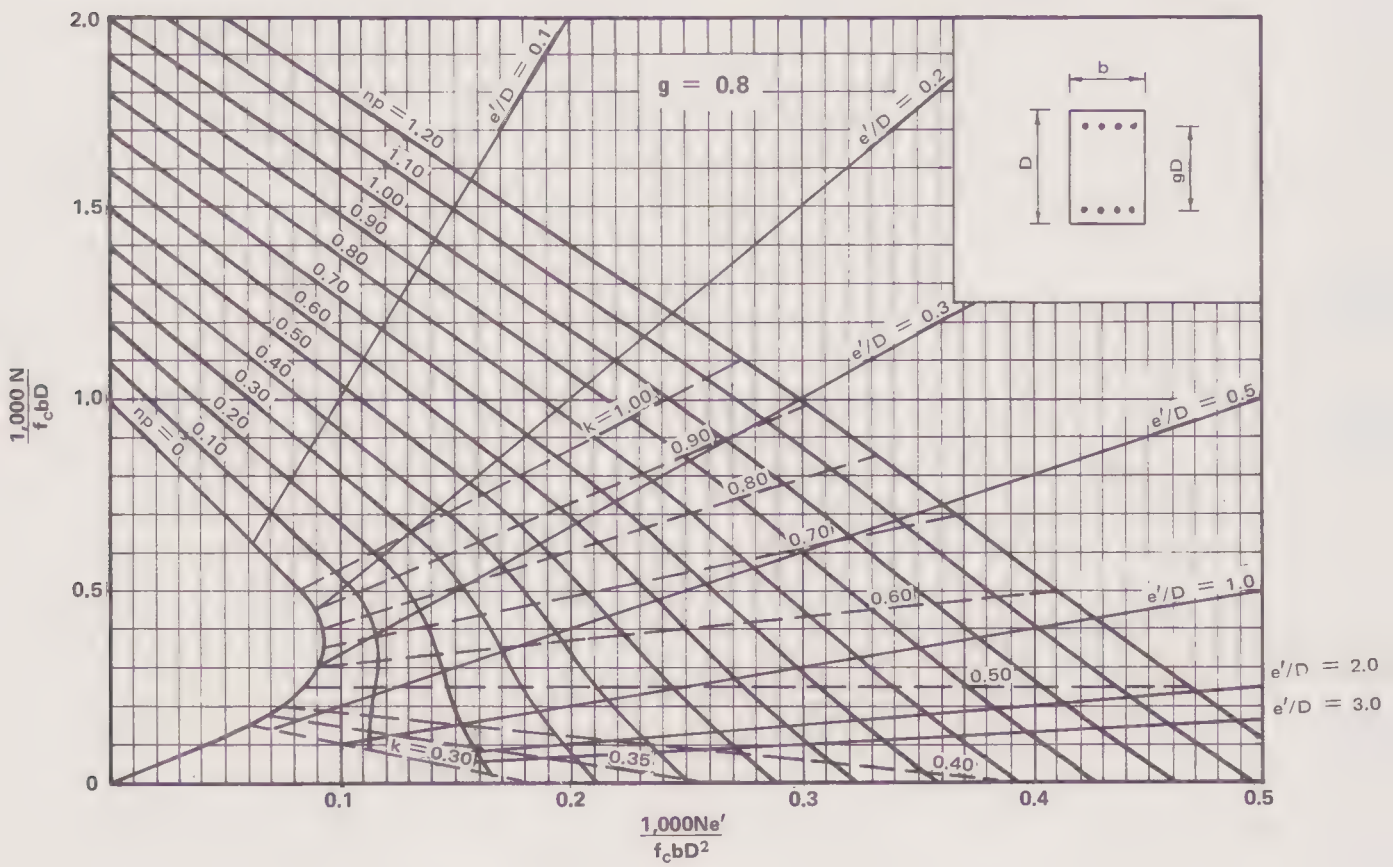
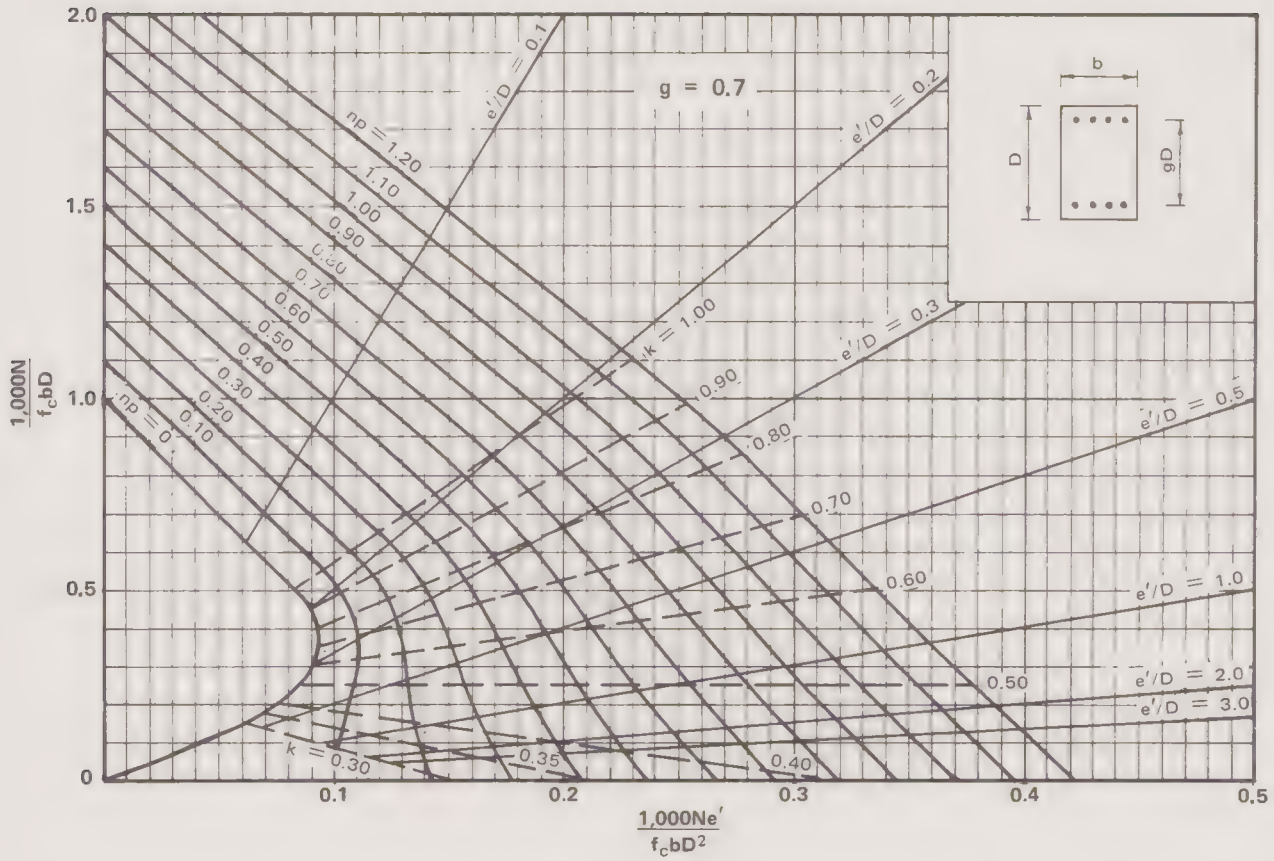


Diagram 21 (ctd.)



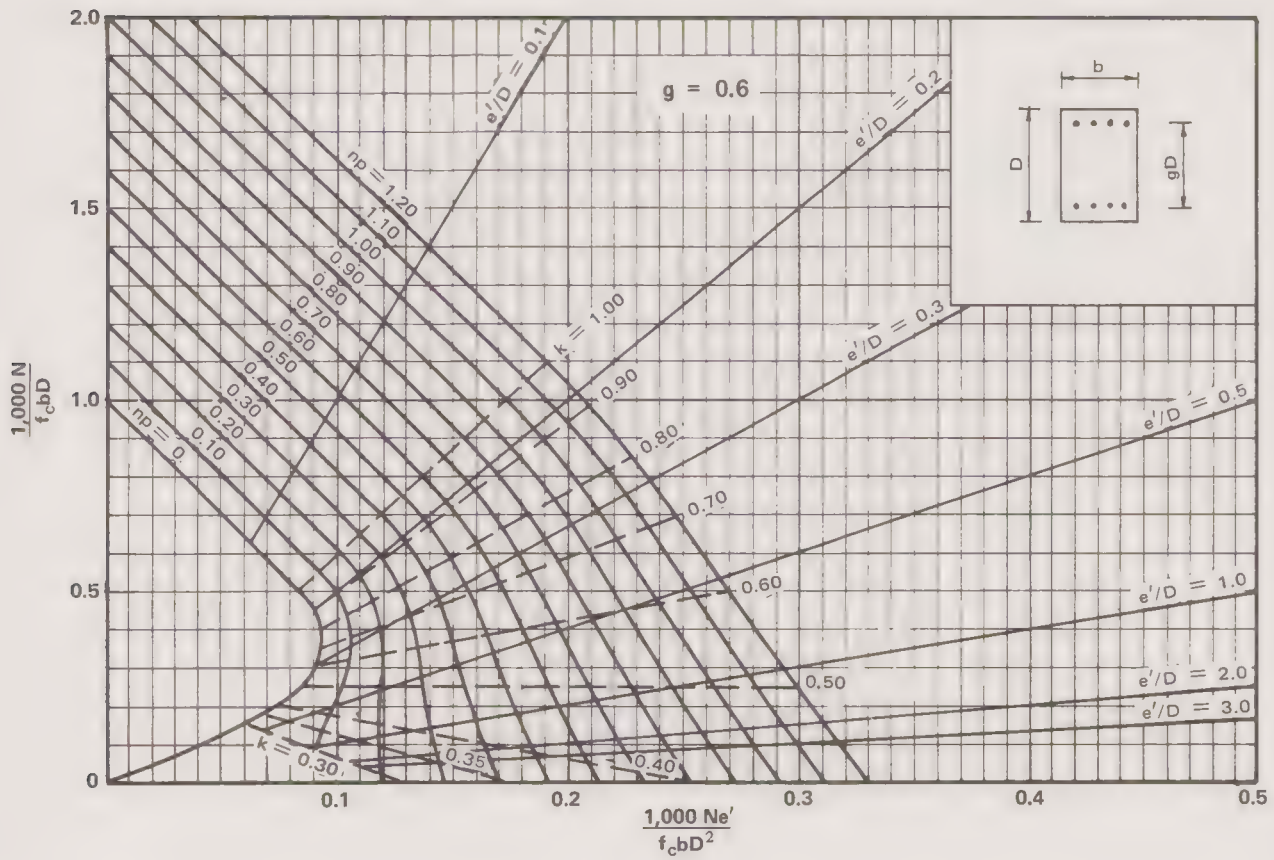
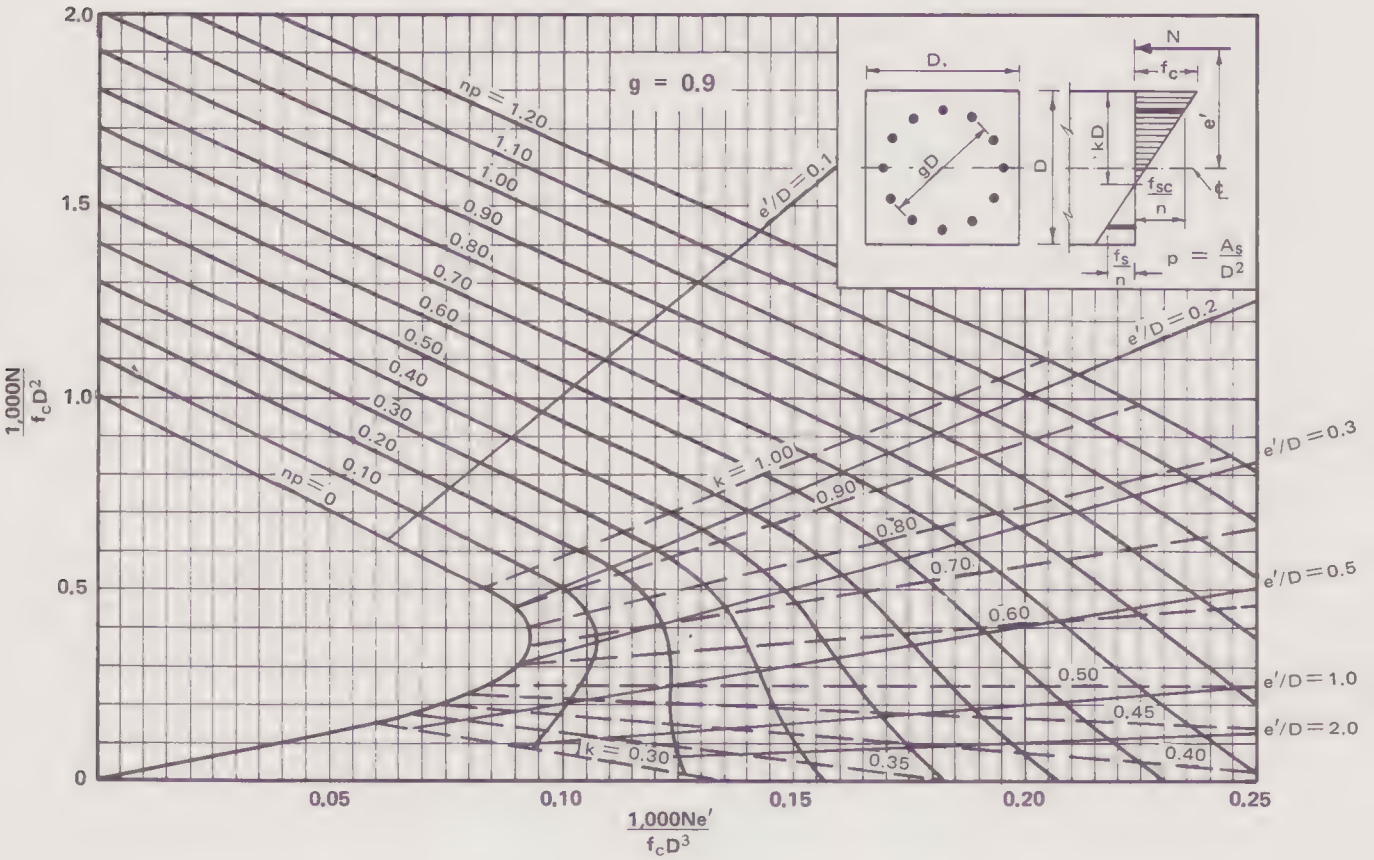
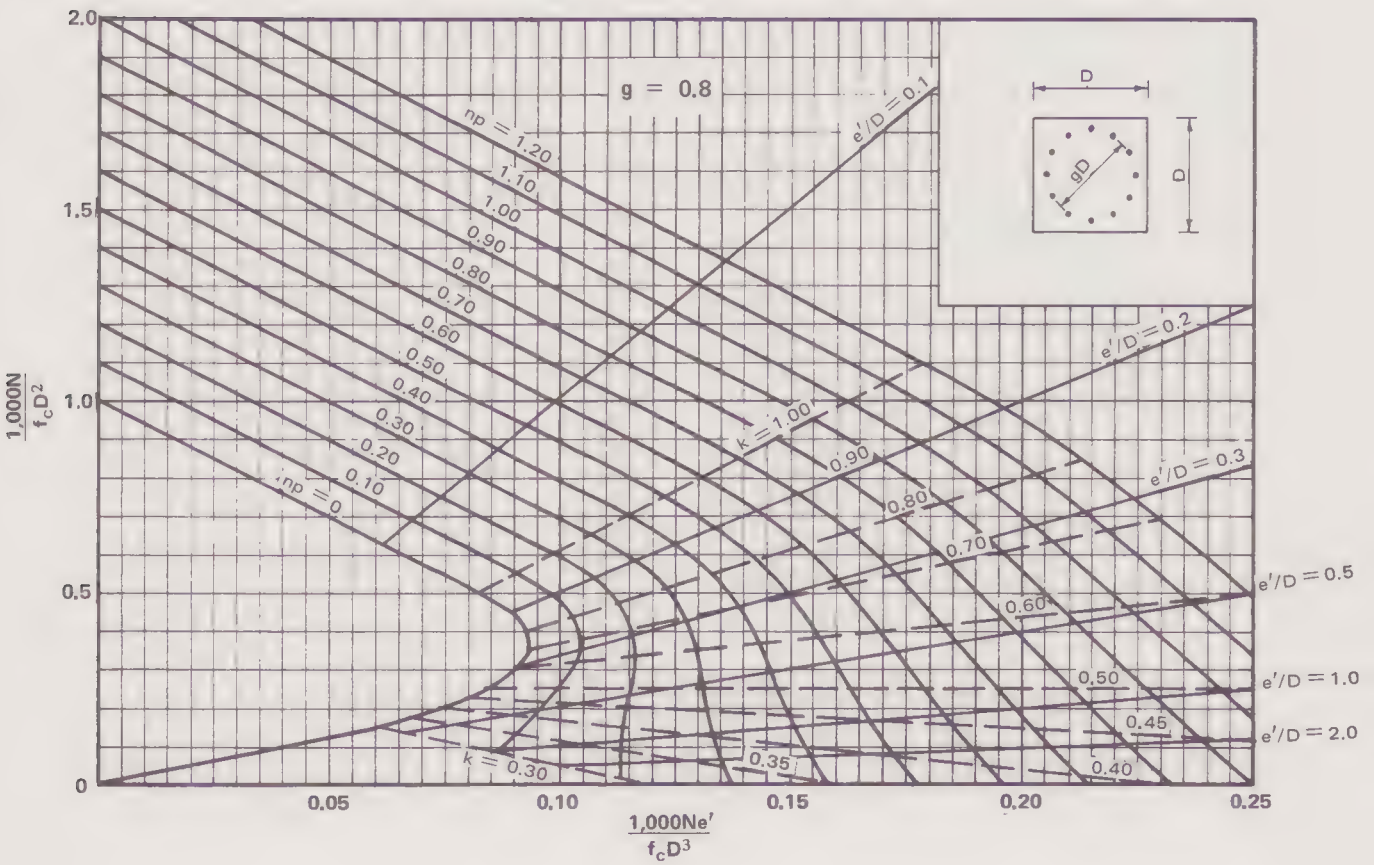
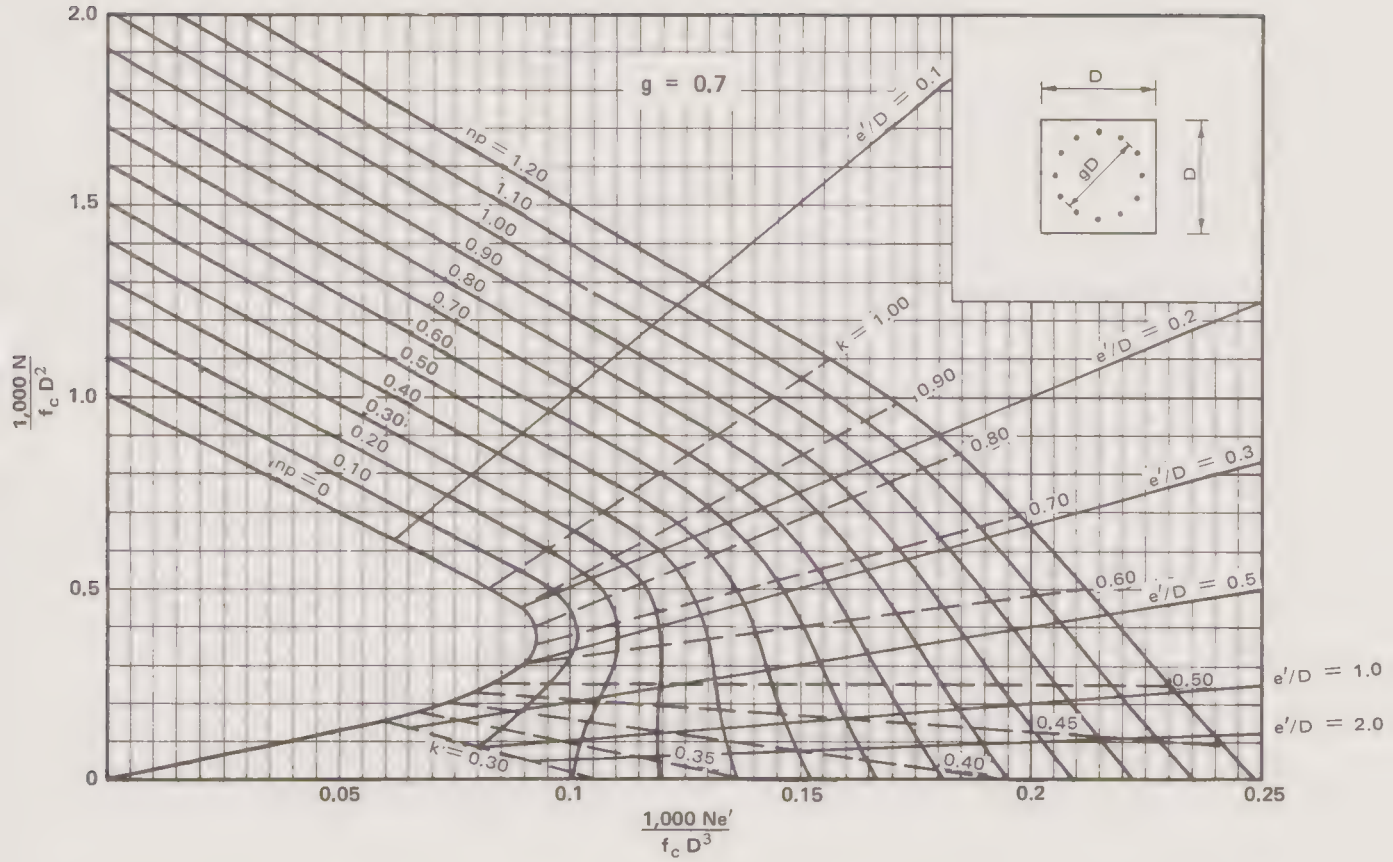


Diagram 22
Eccentrically Loaded Square Compression Members with Helical Reinforcement






g = 0.7
D
gD
D
np = 1.20
1.10
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
np = 0
e'/D = 0.1
k = 1.00
0.90
0.80
0.70
0.60
0.50
0.45
0.40
0.35
k = 0.30
e'/D = 0.2
e'/D = 0.3
e'/D = 0.5
e'/D = 1.0
e'/D = 2.0
2.0
1.5
1.0
0.5
0
0.05
0.1
0.15
0.2
0.25
1,000 N / f_c D^2
1,000 Ne' / f_c D^3

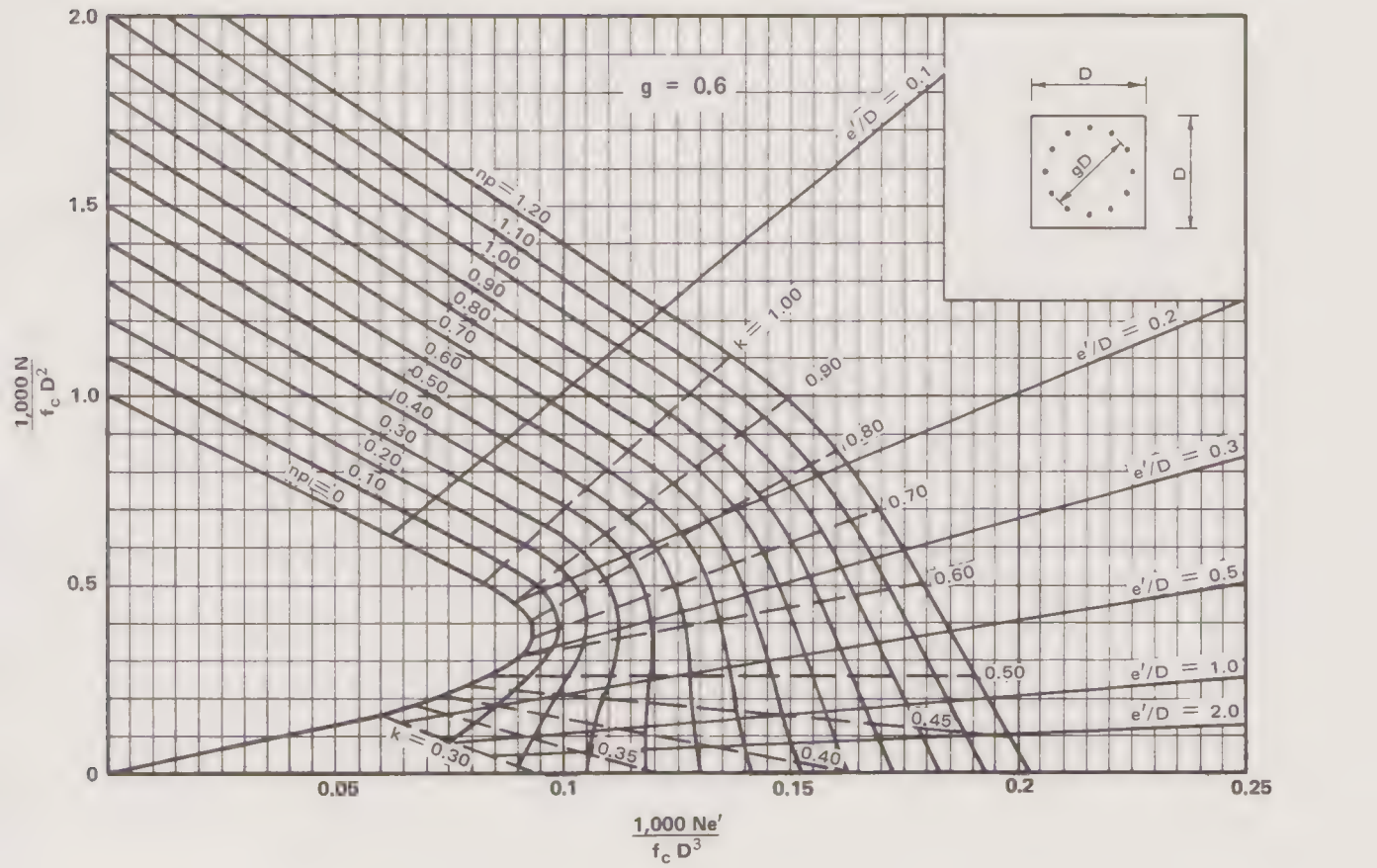
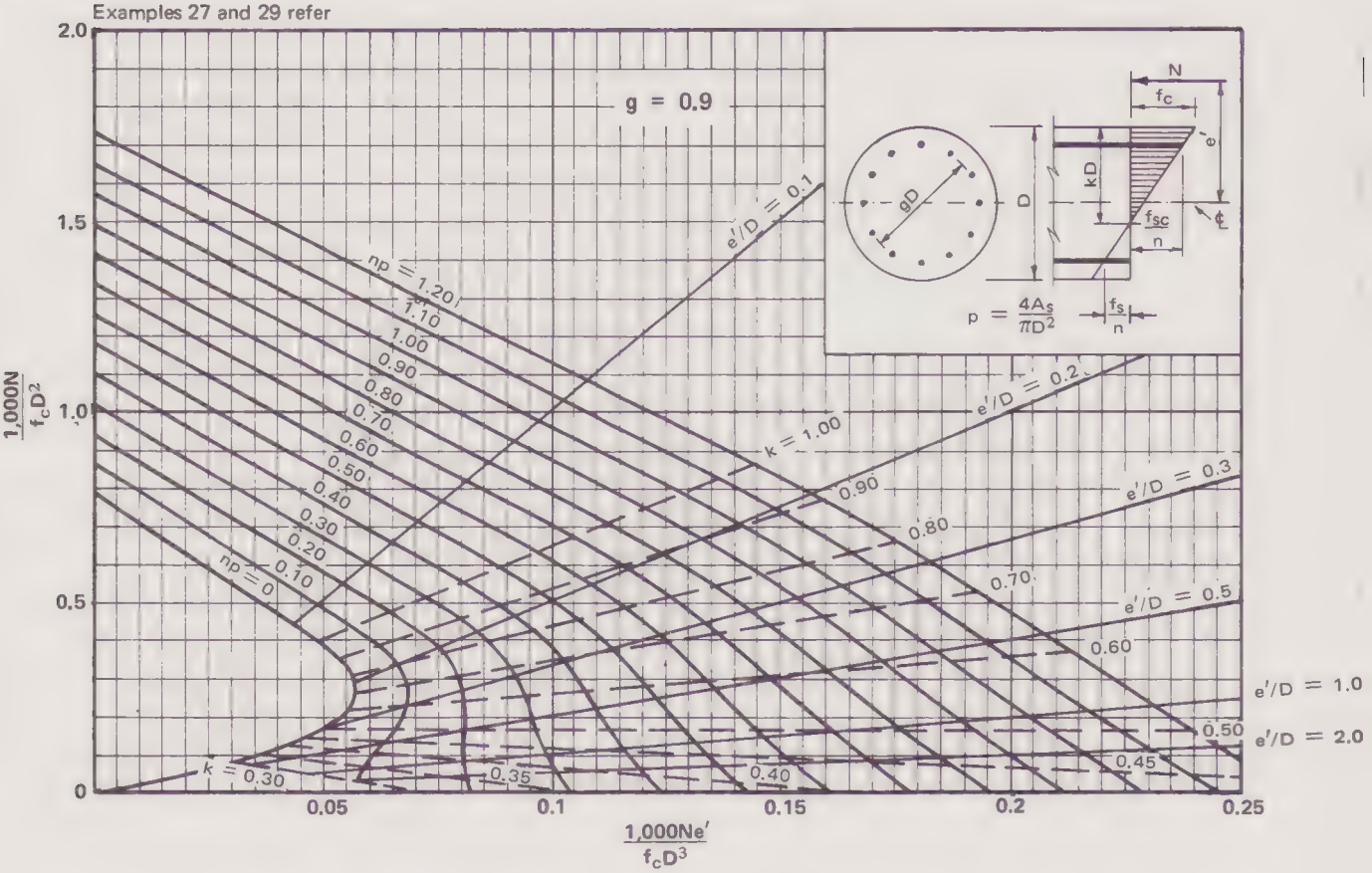

g = 0.6
D
gD
D
np = 1.20
1.10
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
np = 0
e'/D = 0.1
k = 1.00
0.90
0.80
0.70
0.60
0.50
0.45
0.40
0.35
k = 0.30
e'/D = 0.2
e'/D = 0.3
e'/D = 0.5
e'/D = 1.0
e'/D = 2.0
2.0
1.5
1.0
0.5
0
0.05
0.1
0.15
0.2
0.25
1,000 N / f_c D^2
1,000 Ne' / f_c D^3

Diagram 23
Eccentrically Loaded Round Compression Members with Helical Reinforcement

Examples 27 and 29 refer



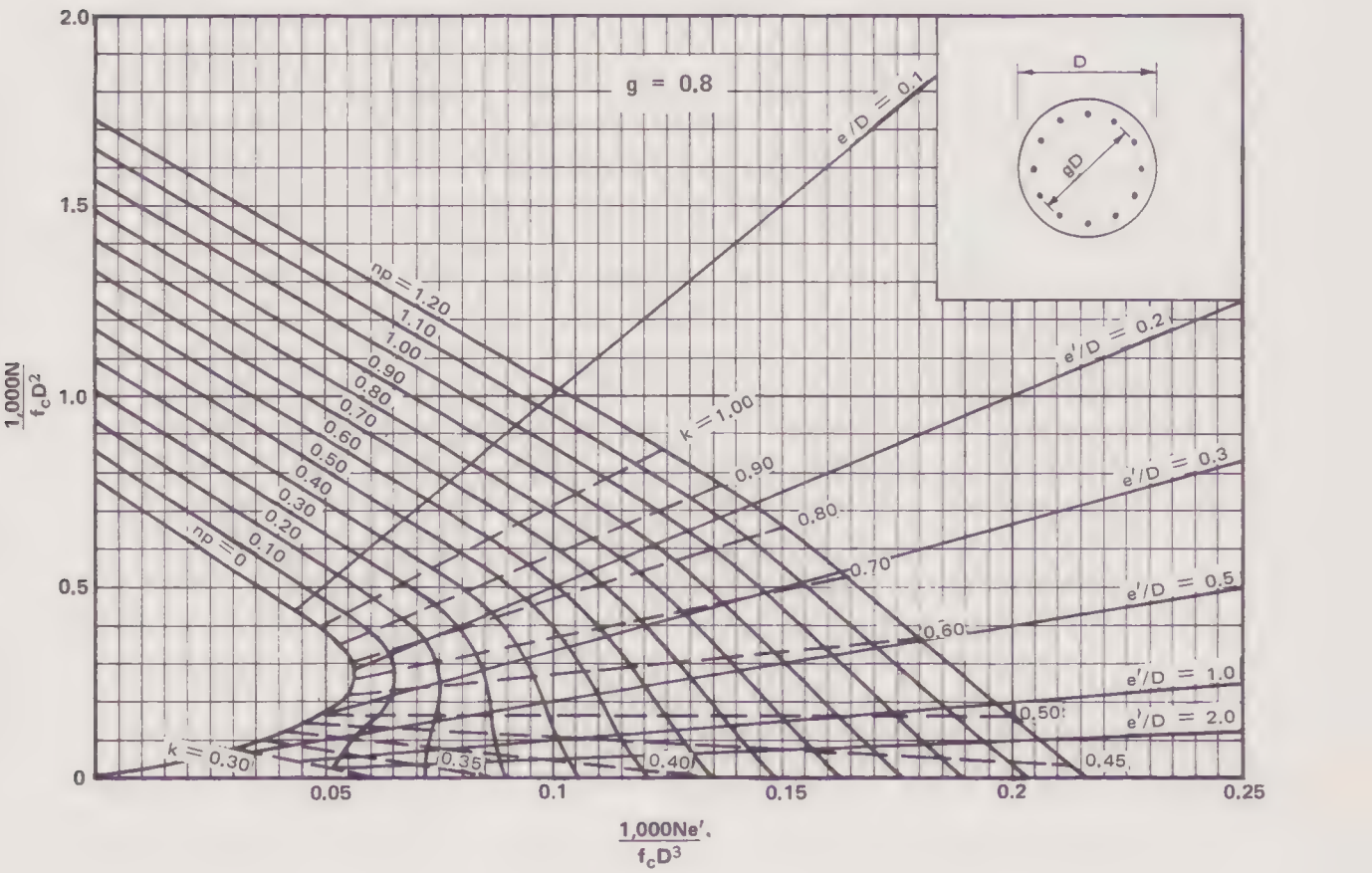
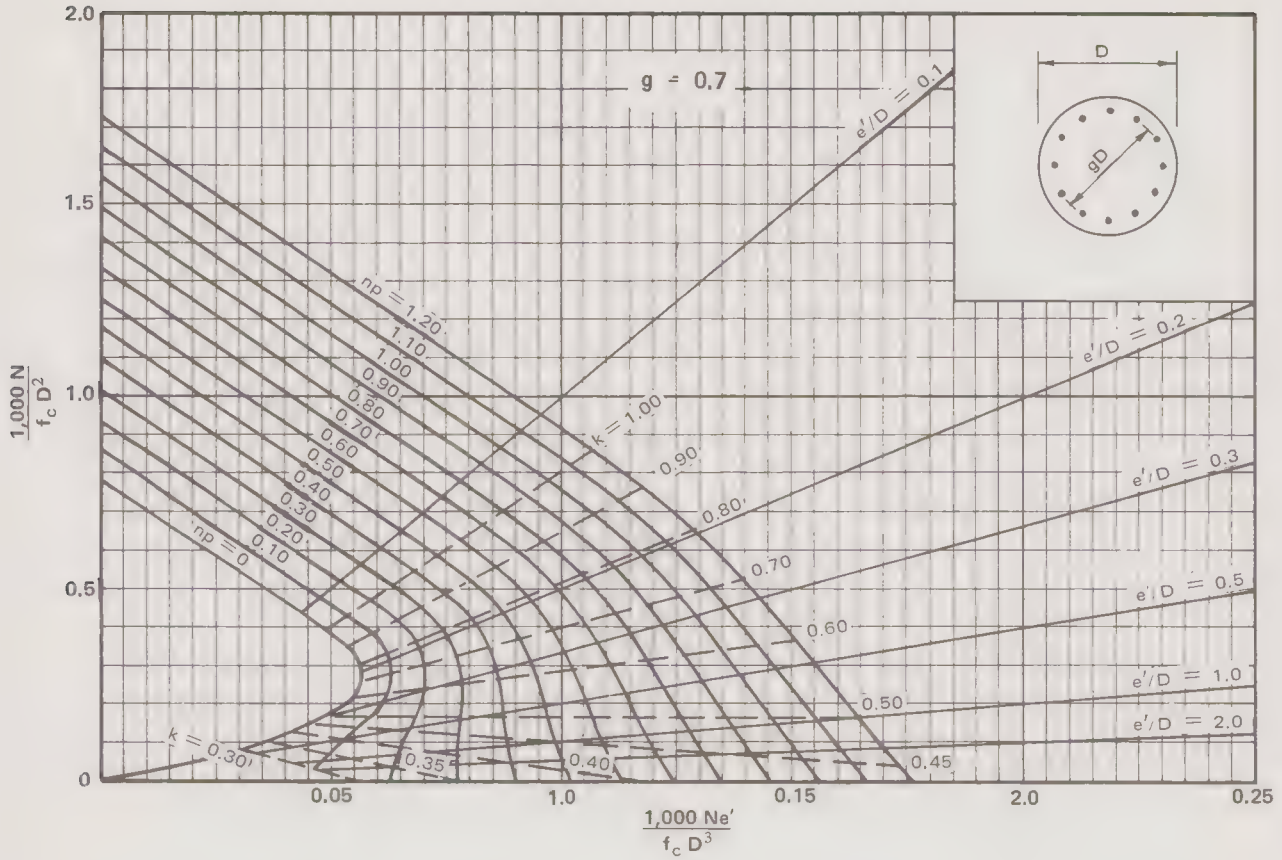
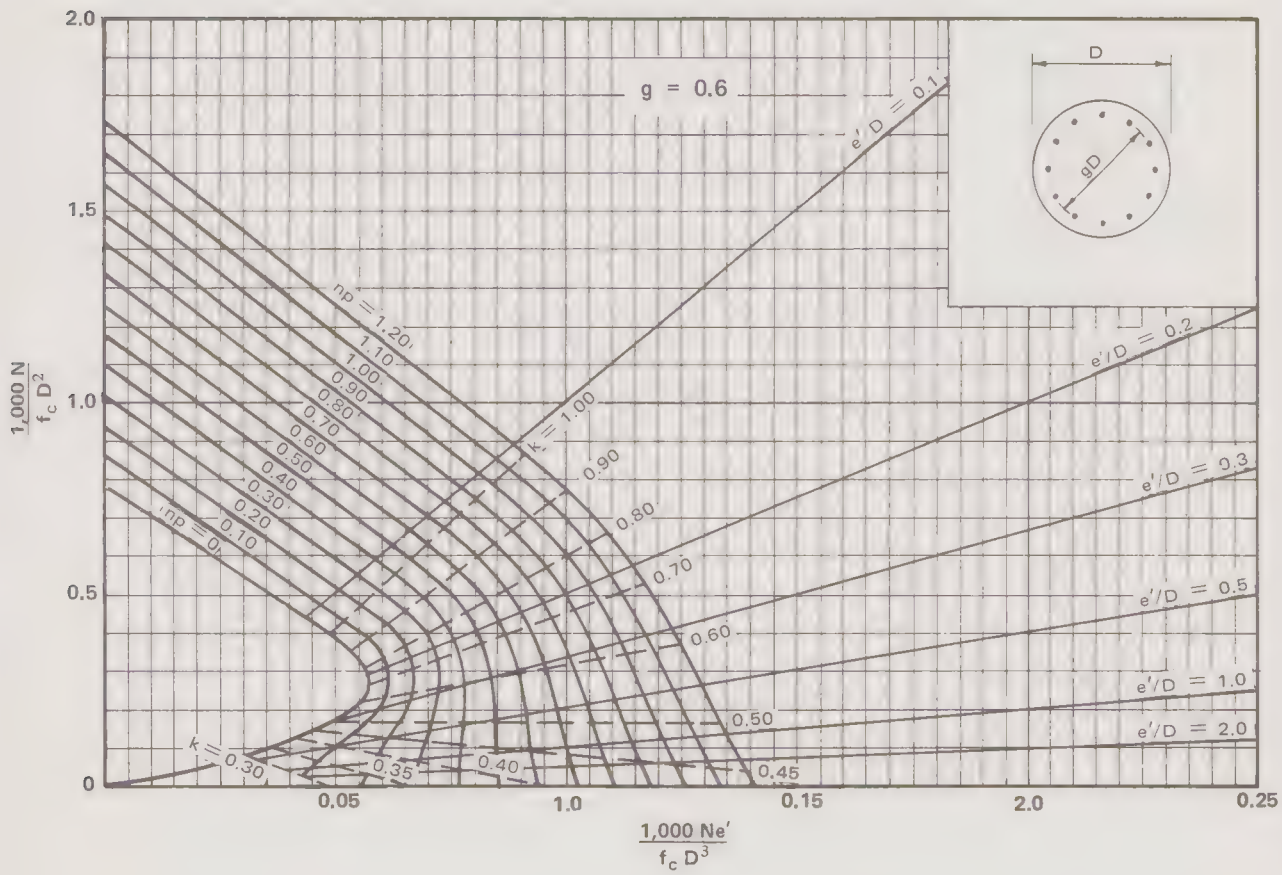


Diagram 23 (ctd.)





Design of Footings

Reinforced Square Spread Footings

Reinforced square spread footings may be readily designed with the aid of Table 24, which for known column loading and soil bearing pressure, gives values of footing width, effective depth, and area and maximum diameter of reinforcement in each direction. Two loadings are tabulated, namely a total load and a footing weight, in order that the effect of the latter may be either ignored or included in the calculation, as demonstrated in Example 30. The information given in the table is accurate for footing weight ignored, and therefore slightly conservative for footing weight included.

The effective depths of all footings given in Table 24 are controlled by maximum shear stress requirements in accordance with AS CA2—1963, except where a practical minimum effective depth of six inches has been imposed; flexural concrete stresses are nowhere critical. The dimension c is a probable minimum size for a square tied concrete column which can support the given total load without exceeding the permissible bearing stress on the footing.

Reinforcement details in the table are based upon 0.85 of the actual moment and shear at the critical section at the face of the assumed minimum column, for $F_s = 20{,}000$ psi. Values of A_s given in light type are controlled by moment and those in heavy type by the bond strength of # 4 bars, the assumed minimum deformed bar size.

Plain Rectangular Footings

The required depth of plain rectangular footings may be obtained from Table 25 when the maximum outstand or cantilever length of the footing from the face of the supported column is known. The table may be used in two ways, as follows:

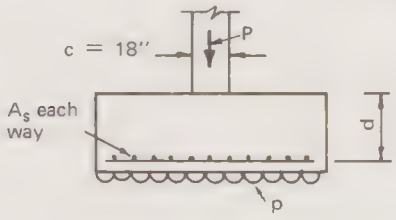
(i) when the footing weight is included in the computations, the value of D is read directly;

(ii) when the footing weight is ignored, the coefficient β is read and D computed from $D = \beta x$.

Both procedures are demonstrated in Example 31.

Example 30
Reinforced Square Spread Footing

A given square column supports a concentric load P and rests on a square reinforced footing. For a maximum permissible bearing pressure p beneath the footing, determine a suitable footing size and reinforcement, assuming that the self weight of the footing is (A) included in the calculation, and (B) ignored.



Given:
F_s = 20,000 psi.
F_c' = 2,500 psi.
p = 6 kips per sq. ft.
P = 560 kips.
Column size 18 in. x 18 in.

(A) Footing weight included

When the self weight of the footing is to be included in the calculation, the permissible soil bearing pressure of 6 kips per sq. ft. must not be exceeded under the combined column loading plus footing weight.

(1) Estimate the footing to be 10.0 ft. square. From Table 24, for F_s/F_c' = 20,000/2,500 and p = 6 kips per sq. ft.:

total capacity of footing = 600 kips,
and self weight of footing* = 36 kips.
Therefore permitted column load = 600 − 36
= 564 kips.
Actual column load P = 560 kips.

Therefore 10 ft. = 120 in. square footing will be satisfactory.

(2) From Table 24: average effective depth d = 25.2 in.;
A_s = 13.0 sq. in.;
and maximum size of deformed bar = #9.

These data are based on an assumed column dimension of 17 in. and therefore will be slightly conservative for the given column which is 18 in. square.

Allowing $2\frac{5}{8}$ in. cover to reinforcement,** total footing depth
= 25.2 + 2.6 + 1.1
= 28.9, say 29 in.

Therefore use footing 120 in. square x 29 in. deep, with 17 #8 bars each way (A_s = 13.4 sq. in.).†

(3) The footing depth must now be checked against the bond length requirement for dowel bars into the column, and the bearing stress on the concrete footing beneath the column must be checked (see AS CA2–1963). These checks are omitted here.

---

* This is the weight of the footing of total depth D = (d + max. deformed bar size + min. cover), rounded to next highest integer; in this case, D = 29 in.

** Min. cover = the greater of 1 in. + $1\frac{1}{2}$ in., or bar dia. + $1\frac{1}{2}$ in. (AS CA2–1963). Cover here corresponds to assumed use of #9 bars.

† The final choice of #8 instead of #9 bars, will reduce the necessary minimum cover and total depth of footing, but not significantly.

Example 30 (ctd.)

(B) Footing weight ignored

When the footing weight is ignored the total soil bearing capacity under the footing may be equated directly to the column load.

(1) From Table 24, for F_s/F'_c = 20,000/2,500, p = 6 kips per sq. ft., and total load = column load = 560 kips : by interpolation,

minimum footing width required = 9.75 ft., say,
= 117 in.
average effective depth d = 24.6 in.;
A_s = 12.4 sq. in.;
and maximum size of deformed bar = #9.

These data are based on an assumed column size of 16½ in. (by interpolation), and therefore will be slightly conservative for the given column which is 18 in. square.

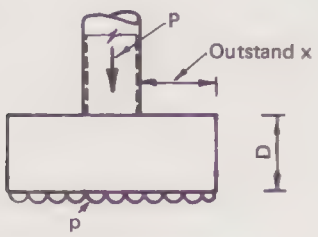
(2) Choosing 16 #8 bars each way (A_s = 12.6 sq. in.), minimum cover = 2½ in. and total footing depth required = 24.6 + 2.5 + 1.0
= 28.1, say 29 in.

Therefore use footing 117 in. square x 29 in. deep, with 16 #8 bars each way.

(3) The additional checks indicated in Section (A)(3) above are also necessary here.

Example 31
Unreinforced Mass Footing

A given round column supports a concentric load P and rests on a square unreinforced mass footing. For a maximum permissible bearing pressure p beneath the footing determine a suitable footing size, assuming that the self weight of the footing is (A) included in the calculation, and (B) ignored.



Given:
F'_c = 2,500 psi.
p = 15 kips per sq. ft.
P = 600 kips.
Column size 21 in. dia.

(A) Footing weight included

(1) Compute side dimension of equivalent square column (AS CA2–1963).

$$\text{side} = \sqrt{\frac{\pi}{4}} \times \text{dia.} = \sqrt{\frac{\pi}{4}} \times 21$$
$$= 18.6, \text{ say } 18\tfrac{1}{2} \text{ in.}$$

(2) Estimate self weight of footing, say 40 kips.

$$\text{Column load} = 600 \text{ kips.}$$
$$\text{Therefore total load} = 640 \text{ kips.}$$
$$\text{Footing area required} = \frac{640}{15}$$
$$= 42.7 \text{ sq. ft.}$$
$$\text{Therefore footing side demension} = \sqrt{42.7}$$
$$= 6.54 \text{ ft., say } 79 \text{ in.}$$
$$\text{Compute outstand } x = \frac{79 - 18\frac{1}{2}}{2}$$
$$= 30\tfrac{1}{4} \text{ in., say } 30 \text{ in.}$$

(3) From Table 25, for $F'_c = 2{,}500$ psi., $p = 15$ kips per sq. ft. and $x = 30$ in.:

$$D = 61.7 \text{ in., say } 62 \text{ in.}$$

$$\text{Check footing weight} = \frac{62 \times 79 \times 79}{12^3} \times 0.150 = 34 \text{ kips.}$$

The initially estimated weight was therefore slightly conservative, but not sufficiently to warrant repetition of the computation. Therefore use footing 79 in. x 79 in. x 62 in. deep.*

(B) Footing weight ignored

(1) Compute dimension of equivalent square column $= 18\frac{1}{2}$ in. as before.

(2)

$$\text{Column load} = 600 \text{ kips.}$$
$$\text{Footing area required} = \frac{600}{15}$$
$$= 40.0 \text{ sq. ft.}$$
$$\text{Therefore footing side dimension} = \sqrt{40.0}$$
$$= 6.32 \text{ ft., say } 76 \text{ in.}$$
$$\text{Compute outstand } x = \frac{76 - 18\frac{1}{2}}{2}$$
$$= 28\tfrac{3}{4}.$$

(3) From Table 25, for $F'_c = 2{,}500$ psi. and $p = 15$ kips per sq. ft.:

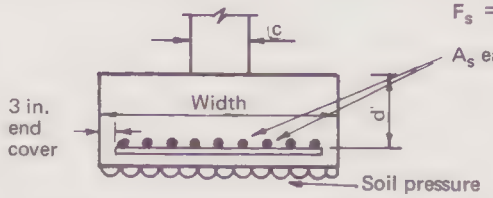
$$\beta = 2.11.$$
$$\text{Therefore required } D = \beta x$$
$$= 2.11 \times 28\tfrac{3}{4}$$
$$= 60.7, \text{ say } 61 \text{ in.}$$

Use footing 76 in. x 76 in. x 61 in. deep.*

* The footing should also be checked for bearing stress on the concrete beneath the column, and for bond length requirement for dowel bars into the column. These checks are omitted here. The vertical compressive stress in the footing need not be checked except when $p = 60$ ksf. or more, as indicated in Table 25.

Table 24
Reinforced Square Spread Footings

1 ksf.



F_s = 20,000 psi.

A_s each way

Note: Table values do not allow for self weight of footings.

Permissible Stresses in Concrete:

F'_c	2,000	2,500	3,000	3,500
n	15	13	12	11
F'_c	900	1,125	1,350	1,575
F_v	60	70	80	90
U plain bar	105	118	131	144
U def. bar	210	236	262	262
U_{av} plain bar	60	67.5	75	82.5
U_{av} def. bar	120	135	150	165

Soil Pressure: 1 kip per sq. ft.

Ftg. width ft.	Total load kips	Ftg. wt. kips	c in.	d in.	A_s sq. in.	Max. bar size plain	Max. bar size def.
			F'_c = 2,000 psi.				
3.0	9	1	9	6.0	0.37		#4
3.5	12	1	9	6.0	0.50		4
4.0	16	2	9	6.0	0.66		4
4.5	20	2	9	6.0	0.83		4
5.0	25	3	9	6.0	1.11		4
5.5	30	4	9	6.1	1.49		4
6.0	36	5	9	6.9	1.77		5
6.5	42	6	9	7.6	2.08		5
7.0	49	7	9	8.3	2.42	#3	6
7.5	56	9	9	9.1	2.77	3	6
8.0	64	11	9	9.8	3.16	3	7
8.5	72	13	9	10.5	3.56	3	7
9.0	81	15	9	11.3	3.99	4	8
9.5	90	18	9	12.0	4.45	4	8
10.0	100	21	9	12.8	4.93	4	9
10.5	110	25	9	13.5	5.44	4	9
11.0	121	29	9	14.3	5.96	5	10
11.5	132	31	9	15.0	6.52	5	10
12.0	144	38	9	15.8	7.10	5	11
12.5	156	41	9	16.5	7.70	5	11
13.0	169	46	9	17.2	8.33	6	11
13.5	182	52	10	17.8	8.98	6	11
14.0	196	56	10	18.5	9.66	6	11
14.5	210	63	10	19.2	10.4	6	11
15.0	225	69	11	19.8	11.1	7	11
			F'_c = 3,000 psi.				
3.0	9	1	9	6.0	0.30		#4
3.5	12	1	9	6.0	0.41		4
4.0	16	2	9	6.0	0.54		4
4.5	20	2	9	6.0	0.79		4
5.0	25	3	9	6.0	1.13		5
5.5	30	3	9	6.0	1.52		5
6.0	36	5	9	6.0	2.07	#3	6
6.5	42	5	9	6.5	2.48	3	7
7.0	49	7	9	7.1	2.87	3	7
7.5	56	8	9	7.8	3.29	4	8
8.0	64	10	9	8.4	3.74	4	9
8.5	72	12	9	9.1	4.21	4	9
9.0	81	14	9	9.7	4.72	5	10
9.5	90	17	9	10.4	5.25	5	11
10.0	100	20	9	11.0	5.81	5	11
10.5	110	22	9	11.7	6.40	6	11
11.0	121	26	9	12.3	7.02	6	11
11.5	132	30	9	13.0	7.66	6	11
12.0	144	32	9	13.6	8.34	7	11
12.5	156	37	9	14.3	9.04	7	11
13.0	169	42	9	15.0	9.77	7	11
13.5	182	46	9	15.6	10.5	8	11
14.0	196	51	9	16.3	11.3	8	11
14.5	210	58	9	16.9	12.1	8	11
15.0	225	62	10	17.3	13.0	8	11

Total load kips	Ftg. wt. kips	c in.	d in.	A_s sq. in.	Max. bar size plain	Max. bar size def.	Ftg. width ft.
		F'_c = 2,500 psi.					
9	1	9	6.0	0.33		#4	3.0
12	1	9	6.0	0.45		4	3.5
16	2	9	6.0	0.59		4	4.0
20	2	9	6.0	0.78		4	4.5
25	3	9	6.0	1.12		4	5.0
30	3	9	6.0	1.52		5	5.5
36	5	9	6.3	1.95		5	6.0
42	6	9	7.0	2.29	#3	6	6.5
49	7	9	7.7	2.65	3	7	7.0
56	8	9	8.3	3.04	3	7	7.5
64	10	9	9.0	3.45	4	8	8.0
72	13	9	9.7	3.90	4	8	8.5
81	15	9	10.4	4.37	4	9	9.0
90	17	9	11.1	4.86	4	9	9.5
100	20	9	11.8	5.38	5	10	10.0
110	24	9	12.5	5.93	5	11	10.5
121	27	9	13.2	6.51	5	11	11.0
132	31	9	13.9	7.11	6	11	11.5
144	34	9	14.6	7.73	6	11	12.0
156	39	9	15.3	8.39	6	11	12;5
169	44	9	16.0	9.07	6	11	13.0
182	48	9	16.7	9.78	7	11	13.5
196	54	10	17.1	10.5	7	11	14.0
210	60	10	17.8	11.3	7	11	14.5
225	65	10	18.5	12.1	8	11	15.0
		F'_c = 3,500 psi.					
9	1	9	6.0	0.27		#4	3.0
12	1	9	6.0	0.38		4	3.5
16	2	9	6.0	0.53		4	4.0
20	2	9	6.0	0.80		4	4.5
25	3	9	6.0	1.14		5	5.0
30	3	9	6.0	1.53	#3	5	5.5
36	4	9	6.0	2.08	3	6	6.0
42	5	9	6.1	2.67	3	7	6.5
49	7	9	6.7	3.08	4	7	7.0
56	8	9	7.3	3.53	4	8	7.5
64	10	9	7.9	4.01	5	9	8.0
72	12	9	8.5	4.52	5	9	8.5
81	14	9	9.1	5.06	5	10	9.0
90	17	9	9.8	5.62	6	11	9.5
100	19	9	10.4	6.22	6	11	10.0
110	22	9	11.0	6.85	6	11	10.5
121	24	9	11.6	7.51	7	11	11.0
132	28	9	12.2	8.20	7	11	11.5
144	32	9	12.9	8.92	7	11	12.0
156	35	9	13.5	9.66	8	11	12.5
169	40	9	14.1	10.4	8	11	13.0
182	43	9	14.7	11.3	8	11	13.5
196	49	9	15.3	12.1	9	11	14.0
210	55	9	16.0	13.0	9	11	14.5
225	59	10	16.4	13.9	9	11	15.0

Example 30 refers

For notes pertaining to this table, see page 135.

continued

Soil Pressure: 2 kips per sq. ft.

Ftg. width ft.	Total load kips	Ftg. wt. kips	c in.	d in.	A_s* sq. in.	Max. bar size plain	Max. bar size def.
			F'_c = 2,000 psi.				
3.0	18	1	9	6.0	**0.74**		#4
3.5	25	1	9	6.0	**1.00**		4
4.0	32	2	9	6.0	**1.31**		4
4.5	41	3	9	7.0	**1.42**		4
5.0	50	3	9	8.0	1.67		4
5.5	61	5	9	9.0	2.03		4
6.0	72	6	9	10.0	2.43		5
6.5	85	8	9	11.0	2.87		5
7.0	98	10	9	12.0	3.34	ϕ3	6
7.5	113	12	9	13.1	3.85	3	6
8.0	128	14	9	14.1	4.40	3	7
8.5	145	17	9	15.1	4.97	3	7
9.0	162	20	9	16.1	5.59	4	8
9.5	181	24	10	16.9	6.22	4	8
10.0	200	28	10	17.9	6.90	4	9
10.5	221	32	11	18.7	7.59	4	9
11.0	242	36	11	19.7	8.35	5	10
11.5	265	41	12	20.5	9.11	5	10
12.0	288	47	12	21.5	9.94	5	11
12.5	313	52	13	22.3	10.8	5	11
13.0	338	59	13	23.3	11.7	6	11
13.5	365	66	14	24.1	12.6	6	11
14.0	392	74	14	25.1	13.5	6	11
14.5	421	81	15	25.9	14.5	6	11
15.0	450	90	15	26.9	15.5	6	11
			F'_c = 3,000 psi.				
3.0	18	1	9	6.0	**0.60**		#4
3.5	25	1	9	6.0	**0.82**		4
4.0	32	2	9	6.0	**1.07**		4
4.5	41	3	9	6.0	1.58		4
5.0	50	3	9	6.9	1.96		5
5.5	61	4	9	7.8	2.39		5
6.0	72	5	9	8.7	2.85	ϕ3	6
6.5	85	7	9	9.6	3.36	3	7
7.0	98	9	9	10.5	3.91	3	7
7.5	113	11	9	11.4	4.50	4	8
8.0	128	14	9	12.3	5.13	4	9
8.5	145	15	9	13.2	5.80	4	9
9.0	162	19	9	14.1	6.51	5	10
9.5	181	22	9	15.0	7.26	5	11
10.0	200	26	9	15.9	8.06	5	11
10.5	221	29	10	16.6	8.87	6	11
11.0	242	33	10	17.5	9.75	6	11
11.5	265	38	10	18.4	10.7	6	11
12.0	288	43	11	19.0	11.6	6	11
12.5	313	49	11	19.9	12.6	7	11
13.0	338	53	12	20.6	13.6	7	11
13.5	365	59	12	21.5	14.7	7	11
14.0	392	66	13	22.2	15.8	8	11
14.5	421	74	13	23.1	16.9	8	11
15.0	450	79	14	23.7	18.1	8	11

Total load kips	Ftg. wt. kips	c in.	d in.	A_s* sq. in.	Max. bar size plain	Max. bar size def.	Ftg. width ft.
			F'_c = 2,500 psi.				
18	1	9	6.0	**0.66**		#4	3.0
25	1	9	6.0	**0.90**		4	3.5
32	2	9	6.0	**1.18**		4	4.0
41	3	9	6.4	1.46		4	4.5
50	3	9	7.4	1.82		4	5.0
61	5	9	8.3	2.21		5	5.5
72	6	9	9.3	2.65		5	6.0
85	7	9	10.2	3.12	ϕ3	6	6.5
98	9	9	11.2	3.63	3	7	7.0
113	11	9	12.1	4.18	3	7	7.5
128	14	9	13.1	4.77	4	8	8.0
145	16	9	14.0	5.40	4	8	8.5
162	19	9	15.0	6.06	4	9	9.0
181	23	9	16.0	6.77	4	9	9.5
200	26	10	16.7	7.48	5	10	10.0
221	30	10	17.6	8.26	5	10	10.5
242	35	11	18.3	9.06	5	11	11.0
265	40	11	19.3	9.91	5	11	11.5
288	45	12	20.0	10.8	6	11	12.0
313	51	12	21.0	11.7	6	11	12.5
338	57	12	21.9	12.7	6	11	13.0
365	62	13	22.6	13.7	7	11	13.5
392	69	13	23.6	14.7	7	11	14.0
421	76	14	24.3	15.8	7	11	14.5
450	84	14	25.2	16.9	7	11	15.0
			F'_c = 3,500 psi.				
18	1	9	6.0	**0.55**		#4	3.0
25	1	9	6.0	**0.77**		4	3.5
32	2	9	6.0	1.06		4	4.0
41	2	9	6.0	1.59		4	4.5
50	3	9	6.5	2.10		5	5.0
61	4	9	7.3	2.56	ϕ3	5	5.5
72	5	9	8.2	3.05	3	6	6.0
85	7	9	9.0	3.59	3	7	6.5
98	9	9	9.9	4.17	4	7	7.0
113	11	9	10.7	4.80	4	8	7.5
128	13	9	11.6	5.47	5	9	8.0
145	15	9	12.5	6.18	5	9	8.5
162	18	9	13.3	6.94	5	10	9.0
181	20	9	14.2	7.74	6	11	9.5
200	25	9	15.0	8.58	6	11	10.0
221	28	10	15.7	9.45	6	11	10.5
242	32	10	16.5	10.4	7	11	11.0
265	36	10	17.4	11.4	7	11	11.5
288	41	11	18.0	12.4	7	11	12.0
313	46	11	18.9	13.4	8	11	12.5
338	51	12	19.5	14.5	8	11	13.0
365	57	12	20.3	15.7	8	11	13.5
392	64	13	21.0	16.8	8	11	14.0
421	71	13	21.8	18.1	9	11	14.5
450	76	14	22.4	19.3	9	11	15.0

continued

Tabulated footing weight based on total depth equal to d + maximum diameter of deformed bar + minimum permissible cover, rounded to next whole inch unit above.

* A_s given in bold type is area for local or average bond strength using #4 bars without hooks. Values of A_s in light type based upon moment requirements. For F_s = 25,000 or 30,000 psi., use the same values of A_s when tabulated in heavy type, assuming #4 bars. Alternatively use $\frac{4}{5}$ or $\frac{2}{3}$ respectively of A_s when tabulated in light type, and check bar size for bond stress.

Soil Pressure: 3 kips per sq. ft.															
Ftg. width ft.	Total load kips	Ftg. wt. kips	c in.	d in.	A_s sq. in.	Max. bar size plain	Max. bar size def.	Total load kips	Ftg. wt. kips	c in.	d in.	A_s sq. in.	Max. bar size plain	Max. bar size def.	Ftg. width ft.
F'_c = 2,000 psi.								F'_c = 2,500 psi.							
3.0	27	1	9	6.0	**1.11**		#4	27	1	9	6.0	**1.00**		#4	3.0
3.5	37	2	9	6.2	**1.47**		4	37	1	9	6.0	**1.35**		4	3.5
4.0	48	2	9	7.4	**1.60**		4	48	2	9	6.8	**1.55**		4	4.0
4.5	61	3	9	8.6	**1.74**		4	61	3	9	8.0	1.77		4	4.5
5.0	75	4	9	9.8	2.04		4	75	4	9	9.1	2.21		4	5.0
5.5	91	5	9	11.0	2.49		4	91	5	9	10.2	2.70		5	5.5
6.0	108	7	9	12.2	2.99		5	108	7	9	11.4	3.24		5	6.0
6.5	127	9	9	13.4	3.54		5	127	8	9	12.5	3.82		6	6.5
7.0	147	11	9	14.6	4.12		6	147	11	9	13.7	4.46		7	7.0
7.5	169	14	9	15.9	4.76		6	169	13	9	14.8	5.14		7	7.5
8.0	192	17	10	16.8	5.40		7	192	16	9	16.0	5.87	ϕ4	8	8.0
8.5	217	20	11	17.8	6.08		7	217	19	10	16.8	6.61	4	8	8.5
9.0	243	23	11	19.0	6.84	ϕ4	8	243	22	11	17.7	7.39	4	9	9.0
9.5	271	27	12	19.9	7.61	4	8	271	26	11	18.9	8.26	4	9	9.5
10.0	300	31	13	20.9	8.41	4	8	300	30	12	19.7	9.14	5	10	10.0
10.5	331	36	13	22.1	9.31	4	9	331	34	12	20.9	10.1	5	10	10.5
11.0	363	41	14	23.0	10.2	4	9	363	41	13	21.8	11.1	5	11	11.0
11.5	397	48	14	24.3	11.2	5	10	397	45	14	22.6	12.1	5	11	11.5
12.0	432	54	15	25.2	12.2	5	10	432	52	14	23.8	13.2	6	11	12.0
12.5	469	60	16	26.2	13.2	5	11	469	56	15	24.7	14.3	6	11	12.5
13.0	507	68	16	27.4	14.3	5	11	507	66	15	25.8	15.5	6	11	13.0
13.5	547	75	17	28.3	15.4	6	11	547	71	16	26.7	16.7	6	11	13.5
14.0	588	83	18	29.3	16.5	6	11	588	81	16	27.8	18.0	7	11	14.0
14.5	631	92	18	30.5	17.7	6	11	631	87	17	28.7	19.2	7	11	14.5
15.0	675	101	19	31.4	19.0	6	11	675	96	18	29.8	20.6	7	11	15.0
F'_c = 3,000 psi.								F'_c = 3,500 psi.							
3.0	27	1	9	6.0	**0.91**		#4	27	1	9	6.0	**0.82**		#4	3.0
3.5	37	1	9	6.0	**1.23**		4	37	1	9	6.0	**1.15**		4	3.5
4.0	48	2	9	6.4	**1.50**		4	48	2	9	6.0	1.58		4	4.0
4.5	61	3	9	7.5	1.90		4	61	3	9	7.1	2.03		4	4.5
5.0	75	4	9	8.6	2.37		5	75	4	9	8.1	2.53		5	5.0
5.5	91	5	9	9.6	2.90		5	91	5	9	9.1	3.09		5	5.5
6.0	108	6	9	10.7	3.47		6	108	6	9	10.1	3.69		6	6.0
6.5	127	8	9	11.8	4.10		7	127	8	9	11.2	4.36		7	6.5
7.0	147	10	9	12.9	4.77		7	147	10	9	12.2	5.08	ϕ4	7	7.0
7.5	169	13	9	14.0	5.50	ϕ4	8	169	12	9	13.2	5.85	4	8	7.5
8.0	192	15	9	15.0	6.28	4	9	192	15	9	14.3	6.68	5	9	8.0
8.5	217	18	9	16.1	7.11	4	9	217	18	9	15.3	7.56	5	9	8.5
9.0	243	21	10	17.0	7.95	5	10	243	21	10	16.1	8.45	5	10	9.0
9.5	271	25	11	17.8	8.84	5	10	271	24	11	16.9	9.40	5	10	9.5
10.0	300	30	11	18.9	9.83	5	11	300	29	11	17.9	10.4	6	11	10.0
10.5	331	33	12	19.7	10.8	5	11	331	32	12	18.7	11.5	6	11	10.5
11.0	363	39	12	20.8	11.9	6	11	363	36	12	19.7	12.6	6	11	11.0
11.5	397	43	13	21.6	13.0	6	11	397	41	13	20.5	13.8	7	11	11.5
12.0	432	49	13	22.7	14.2	6	11	432	47	13	21.5	15.0	7	11	12.0
12.5	469	54	14	23.5	15.3	7	11	469	52	14	22.3	16.3	7	11	12.5
13.0	507	61	14	24.6	16.6	7	11	507	59	14	23.3	17.7	8	11	13.0
13.5	547	68	15	25.4	17.9	7	11	547	66	15	24.1	19.0	8	11	13.5
14.0	588	76	16	26.3	19.2	7	11	588	74	16	24.9	20.4	8	11	14.0
14.5	631	84	16	27.4	20.7	8	11	631	81	16	25.9	22.0	9	11	14.5
15.0	675	93	17	28.2	22.1	8	11	675	87	17	26.7	23.5	9	11	15.0

continued

For notes pertaining to this table, see page 135.

Soil Pressure: 4 kips per sq. ft.																
Ftg. width ft.	Total load kips	Ftg. wt. kips	c in.	d in.	A_s sq. in.	Max. bar size plain	Max. bar size def.	Total load kips	Ftg. wt. kips	c in.	d in.	A_s sq. in.	Max. bar size plain	Max. bar size def.	Ftg. width ft.
	F'_c = 2,000 psi.							F'_c = 2,500 psi.							
3.0	36	1	9	6.0	**1.48**		#4	36	1	9	6.0	**1.33**		#4	3.0
3.5	49	2	9	7.1	**1.70**		4	49	2	9	6.6	**1.63**		4	3.5
4.0	64	2	9	8.4	**1.86**		4	64	2	9	7.9	**1.79**		4	4.0
4.5	81	3	9	9.8	**2.03**		4	81	3	9	9.2	2.05		4	4.5
5.0	100	5	9	11.2	2.38		4	100	4	9	10.4	2.57		4	5.0
5.5	121	6	9	12.5	2.91		4	121	6	9	11.7	3.14		5	5.5
6.0	144	8	9	13.9	3.50		5	144	8	9	13.0	3.77		5	6.0
6.5	169	10	9	15.3	4.14		5	169	10	9	14.3	4.46		6	6.5
7.0	196	12	10	16.4	4.79		6	196	12	10	15.3	5.16		6	7.0
7.5	225	15	11	17.5	5.48		6	225	14	10	16.6	5.95		7	7.5
8.0	256	18	12	18.5	6.23		7	256	18	11	17.6	6.76	ϕ4	8	8.0
8.5	289	22	12	19.9	7.07		7	289	21	12	18.7	7.61	4	8	8.5
9.0	324	25	13	21.0	7.91		7	324	24	12	19.9	8.57	4	9	9.0
9.5	361	29	14	22.1	8.80	ϕ4	8	361	28	13	21.0	9.53	4	9	9.5
10.0	400	34	14	23.5	9.79	4	8	400	33	14	22.0	10.5	5	10	10.0
10.5	441	40	15	24.5	10.8	4	9	441	39	14	23.3	11.7	5	10	10.5
11.0	484	45	16	25.6	11.8	4	9	484	44	15	24.3	12.8	5	11	11.0
11.5	529	51	17	26.7	12.9	5	10	529	50	16	25.3	14.0	5	11	11.5
12.0	576	59	17	28.1	14.1	5	10	576	56	16	26.6	15.2	6	11	12.0
12.5	625	66	18	29.2	15.3	5	11	625	62	17	27.6	16.5	6	11	12.5
13.0	676	74	19	30.3	16.5	5	11	676	70	18	28.6	17.8	6	11	13.0
13.5	729	82	19	31.6	17.8	6	11	729	80	18	29.9	19.3	6	11	13.5
14.0	784	91	20	32.7	19.2	6	11	784	88	19	30.9	20.7	7	11	14.0
14.5	841	103	21	33.8	20.5	6	11	841	97	20	31.9	22.2	7	11	14.5
15.0	900	112	22	34.9	21.9	6	11	900	107	20	33.2	23.8	7	11	15.0
	F'_c = 3,000 psi.							F'_c = 3,500 psi.							
3.0	36	1	9	6.0	**1.21**		#4	36	1	9	6.0	**1.09**		#4	3.0
3.5	49	2	9	6.2	**1.58**		4	49	1	9	6.0	**1.54**		4	3.5
4.0	64	2	9	7.4	**1.73**		4	64	2	9	7.0	1.82		4	4.0
4.5	81	3	9	8.6	2.20		4	81	3	9	8.2	2.33		4	4.5
5.0	100	4	9	9.9	2.75		5	100	4	9	9.3	2.92		5	5.0
5.5	121	6	9	11.1	3.36		5	121	5	9	10.5	3.56		5	5.5
6.0	144	7	9	12.3	4.03		6	144	7	9	11.7	4.27		6	6.0
6.5	169	9	9	13.5	4.76		7	169	9	9	12.8	5.05		7	6.5
7.0	196	12	9	14.8	5.55		7	196	11	9	14.0	5.89	ϕ4	7	7.0
7.5	225	14	10	15.7	6.36	ϕ4	8	225	13	10	14.9	6.74	4	8	7.5
8.0	256	17	10	16.9	7.27	4	9	256	16	10	16.1	7.70	4	9	8.0
8.5	289	20	11	17.9	8.18	4	9	289	19	11	17.0	8.68	5	9	8.5
9.0	324	23	12	18.9	9.15	5	10	324	22	12	17.9	9.71	5	10	9.0
9.5	361	28	12	20.1	10.2	5	10	361	27	12	19.1	10.9	5	10	9.5
10.0	400	33	13	21.0	11.3	5	11	400	31	13	20.0	12.0	6	11	10.0
10.5	441	37	13	22.3	12.5	5	11	441	36	13	21.2	13.3	6	11	10.5
11.0	484	42	14	23.2	13.7	6	11	484	41	14	22.1	14.6	6	11	11.0
11.5	529	48	15	24.2	15.0	6	11	529	46	15	23.0	15.9	7	11	11.5
12.0	576	54	15	25.4	16.4	6	11	576	52	15	24.1	17.3	7	11	12.0
12.5	625	60	16	26.4	17.7	7	11	625	58	16	25.1	18.8	7	11	12.5
13.0	676	68	17	27.3	19.1	7	11	676	66	17	26.0	20.3	8	11	13.0
13.5	729	75	17	28.6	20.7	7	11	729	73	17	27.1	21.9	8	11	13.5
14.0	784	83	18	29.5	22.2	7	11	784	81	18	28.0	23.6	8	11	14.0
14.5	841	92	19	30.5	23.8	8	11	841	89	19	28.9	25.2	8	11	14.5
15.0	900	101	19	31.7	25.5	8	11	900	99	19	30.1	27.1	9	11	15.0

continued

For notes pertaining to this table, see page 135.

Soil Pressure: 5 kips per sq. ft.															
Ftg. width ft.	Total load kips	Ftg. wt. kips	c in.	d in.	A_s sq. in.	Max. bar size plain	Max. bar size def.	Total load kips	Ftg. wt. kips	c in.	d in.	A_s sq. in.	Max. bar size plain	Max. bar size def.	Ftg. width ft.
F'_c = 2,000 psi.								F'_c = 2,500 psi.							
3.0	45	1	9	6.4	**1.75**		#4	45	1	9	6.0	**1.66**		#4	3.0
3.5	61	2	9	7.8	**1.92**		4	61	2	9	7.3	**1.84**		4	3.5
4.0	80	3	9	9.3	**2.11**		4	80	2	9	8.7	**2.02**		4	4.0
4.5	101	4	9	10.8	**2.31**		4	101	4	9	10.1	2.32		4	4.5
5.0	125	5	9	12.3	2.70		4	125	5	9	11.5	2.90		4	5.0
5.5	151	6	9	13.8	3.31		4	151	6	9	13.0	3.55		5	5.5
6.0	180	9	10	15.0	3.93		5	180	8	9	14.4	4.27		5	6.0
6.5	211	11	10	16.5	4.66		5	211	10	10	15.5	5.00		6	6.5
7.0	245	13	11	17.7	5.39		6	245	12	11	16.6	5.78		6	7.0
7.5	281	16	12	18.9	6.18		6	281	15	11	18.0	6.69		7	7.5
8.0	320	19	13	20.1	7.01		6	320	18	12	19.2	7.59		7	8.0
8.5	361	23	14	21.3	7.91		7	361	22	13	20.3	8.55		8	8.5
9.0	405	26	15	22.5	8.85		7	405	25	14	21.4	9.57		8	9.0
9.5	451	32	15	24.0	9.93		8	451	30	14	22.8	10.7		9	9.5
10.0	500	36	16	25.2	11.0		8	500	35	15	23.9	11.9		9	10.0
10.5	551	43	17	26.4	12.1		9	551	41	16	25.1	13.1	ϕ5	10	10.5
11.0	605	48	18	27.6	13.3		9	605	47	17	26.2	14.3	5	10	11.0
11.5	661	55	19	28.8	14.5		9	661	53	17	27.6	15.7	5	11	11.5
12.0	720	63	19	30.3	15.8	ϕ5	10	720	59	18	28.7	17.1	5	11	12.0
12.5	781	70	20	31.5	17.2	5	10	781	68	19	29.9	18.5	6	11	12.5
13.0	845	78	21	32.7	18.5	5	11	845	76	20	31.0	20.0	6	11	13.0
13.5	911	89	22	33.9	20.0	5	11	911	84	20	32.4	21.7	6	11	13.5
14.0	980	98	23	35.1	21.5	6	11	980	93	21	33.5	23.3	6	11	14.0
14.5	1051	108	23	36.6	23.1	6	11	1051	103	22	34.6	24.9	7	11	14.5
15.0	1125	121	24	37.8	24.7	6	11	1125	115	23	35.8	26.6	7	11	15.0
F'_c = 3,000 psi.								F'_c = 3,500 psi.							
3.0	45	1	9	6.0	**1.51**		#4	45	1	9	6.0	**1.37**		#4	3.0
3.5	61	2	9	6.9	**1.77**		4	61	2	9	6.6	**1.76**		4	3.5
4.0	80	2	9	8.3	**1.94**		4	80	2	9	7.8	2.03		4	4.0
4.5	101	3	9	9.6	2.47		4	101	3	9	9.1	2.62		4	4.5
5.0	125	5	9	10.9	3.09		5	125	4	9	10.4	3.28		5	5.0
5.5	151	6	9	12.3	3.79		5	151	6	9	11.7	4.01		5	5.5
6.0	180	8	9	13.6	4.55		6	180	8	9	13.0	4.81		6	6.0
6.5	211	10	9	15.0	5.38		7	211	10	9	14.2	5.69		7	6.5
7.0	245	12	10	16.0	6.22		7	245	12	10	15.3	6.58	ϕ4	7	7.0
7.5	281	15	11	17.1	7.12	ϕ4	8	281	14	11	16.3	7.54	4	8	7.5
8.0	320	18	11	18.4	8.15	4	8	320	18	11	17.6	8.63	4	8	8.0
8.5	361	22	12	19.5	9.18	4	9	361	21	12	18.6	9.71	5	9	8.5
9.0	405	25	13	20.6	10.3	4	9	405	24	13	19.6	10.9	5	9	9.0
9.5	451	29	14	21.6	11.4	5	10	451	28	14	20.6	12.1	5	10	9.5
10.0	500	35	14	23.0	12.7	5	11	500	34	14	21.9	13.5	6	11	10.0
10.5	551	40	15	24.0	14.0	5	11	551	39	15	22.9	14.8	6	11	10.5
11.0	605	45	16	25.1	15.3	6	11	605	44	16	23.9	16.2	6	11	11.0
11.5	661	51	17	26.2	16.7	6	11	661	50	17	24.9	17.7	6	11	11.5
12.0	720	58	17	27.5	18.3	6	11	720	56	17	26.2	19.4	7	11	12.0
12.5	781	64	18	28.6	19.8	6	11	781	62	18	27.2	21.0	7	11	12.5
13.0	845	72	19	29.6	21.4	7	11	845	70	19	28.2	22.7	7	11	13.0
13.5	911	82	19	31.0	23.2	7	11	911	77	19	29.5	24.5	8	11	13.5
14.0	980	91	20	32.1	24.9	7	11	980	86	20	30.5	26.3	8	11	14.0
14.5	1051	100	21	33.1	26.7	8	11	1051	95	21	31.5	28.2	8	11	14.5
15.0	1125	110	22	34.2	28.5	8	11	1125	104	22	32.5	30.2	9	11	15.0

continued

For notes pertaining to this table, see page 135.

Soil Pressure: 6 kips per sq. ft.															
Ftg. width ft.	Total load kips	Ftg. wt. kips	c in.	d in.	A_s sq. in.	Max. bar size plain	Max. bar size def.	Total load kips	Ftg. wt. kips	c in.	d in.	A_s sq. in.	Max. bar size plain	Max. bar size def.	Ftg. width ft.
F'_c = 2,000 psi.								F'_c = 2,500 psi.							
3.0	54	1	9	6.9	**1.95**		#4	54	1	9	6.4	**1.85**		#4	3.0
3.5	74	2	9	8.5	**2.13**		4	74	2	9	7.9	**2.03**		4	3.5
4.0	96	3	9	10.0	**2.35**		4	96	3	9	9.5	**2.23**		4	4.0
4.5	122	4	9	11.6	**2.57**		4	122	4	9	11.0	2.57		4	4.5
5.0	150	5	9	13.2	3.01		4	150	5	9	12.5	3.22		4	5.0
5.5	182	7	10	14.5	3.64		4	182	7	9	14.0	3.95		5	5.5
6.0	216	9	11	15.8	4.33		5	216	9	10	15.2	4.69		5	6.0
6.5	254	11	11	17.4	5.14		5	254	11	11	16.4	5.49		6	6.5
7.0	294	13	12	18.7	5.95		6	294	13	12	17.6	6.36		6	7.0
7.5	338	17	13	20.0	6.82		6	338	16	12	19.2	7.37		7	7.5
8.0	384	20	14	21.3	7.75		6	384	19	13	20.4	8.37		7	8.0
8.5	434	23	15	22.6	8.73		7	434	23	14	21.6	9.43		8	8.5
9.0	486	28	16	23.9	9.78		7	486	27	15	22.8	10.6		8	9.0
9.5	542	33	17	25.2	10.9		8	542	32	16	24.0	11.7		9	9.5
10.0	600	38	18	26.5	12.1		8	600	36	17	25.2	13.0		9	10.0
10.5	662	44	19	27.8	13.3		8	662	43	17	26.8	14.4	ϕ5	10	10.5
11.0	726	51	19	29.4	14.7		9	726	48	18	28.0	15.8	5	10	11.0
11.5	794	58	20	30.7	16.0		9	794	56	19	29.2	17.3	5	11	11.5
12.0	864	65	21	31.9	17.4	ϕ5	10	864	63	20	30.4	18.8	5	11	12.0
12.5	938	74	22	33.2	18.9	5	10	938	70	21	31.6	20.3	6	11	12.5
13.0	1014	82	23	34.5	20.4	5	11	1014	80	22	32.8	22.0	6	11	13.0
13.5	1094	93	24	35.8	22.0	5	11	1094	89	22	34.4	23.8	6	11	13.5
14.0	1176	103	25	37.1	23.7	6	11	1176	98	23	35.6	25.6	6	11	14.0
14.5	1262	113	26	38.4	25.4	6	11	1262	110	24	36.8	27.4	7	11	14.5
15.0	1350	124	27	39.7	27.1	6	11	1350	121	25	38.0	29.3	7	11	15.0
F'_c = 3,000 psi.								F'_c = 3,500 psi.							
3.0	54	1	9	6.1	**1.78**		#4	54	1	9	6.0	**1.64**		#4	3.0
3.5	74	2	9	7.5	**1.95**		4	74	2	9	7.2	**1.93**		4	3.5
4.0	96	2	9	9.0	**2.14**		4	96	2	9	8.5	2.24		4	4.0
4.5	122	4	9	10.4	2.73		4	122	3	9	9.9	2.89		4	4.5
5.0	150	5	9	11.8	3.43		5	150	5	9	11.3	3.62		5	5.0
5.5	182	6	9	13.3	4.20		5	182	6	9	12.7	4.43		5	5.5
6.0	216	8	9	14.7	5.05		6	216	8	9	14.1	5.33		6	6.0
6.5	254	11	10	15.9	5.90		7	254	10	10	15.2	6.23		7	6.5
7.0	294	13	11	17.0	6.83		7	294	12	11	16.3	7.21	ϕ4	7	7.0
7.5	338	15	12	18.2	7.83		8	338	15	12	17.4	8.26	4	8	7.5
8.0	384	18	13	19.4	8.89		8	384	18	13	18.5	9.39	4	8	8.0
8.5	434	23	13	20.8	10.1		9	434	22	13	19.8	10.7	5	9	8.5
9.0	486	26	14	22.0	11.3		9	486	25	14	20.9	11.9	5	9	9.0
9.5	542	32	15	23.1	12.6	ϕ5	10	542	29	15	22.0	13.3	5	10	9.5
10.0	600	36	16	24.3	13.9	5	10	600	35	16	23.1	14.7	5	10	10.0
10.5	662	41	17	25.4	15.3	5	11	662	40	17	24.2	16.2	6	11	10.5
11.0	726	48	17	26.9	16.9	6	11	726	45	17	25.6	17.9	6	11	11.0
11.5	794	55	18	28.0	18.5	6	11	794	51	18	26.7	19.5	6	11	11.5
12.0	864	61	19	29.2	20.1	6	11	864	59	19	27.8	21.2	7	11	12.0
12.5	938	68	20	30.3	21.7	6	11	938	66	20	28.9	23.0	7	11	12.5
13.0	1014	78	20	31.8	23.6	7	11	1014	74	20	30.3	24.9	7	11	13.0
13.5	1094	87	21	32.9	25.4	7	11	1094	82	21	31.4	26.9	8	11	13.5
14.0	1176	96	22	34.1	27.3	7	11	1176	91	22	32.5	28.9	8	11	14.0
14.5	1262	105	23	35.2	29.3	7	11	1262	100	23	33.6	30.9	8	11	14.5
15.0	1350	115	24	36.4	31.3	8	11	1350	110	24	34.7	33.1	8	11	15.0

continued

For notes pertaining to this table, see page 135.

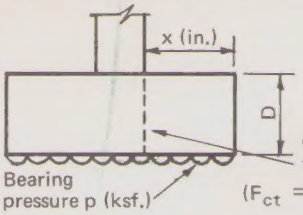
Soil Pressure: 8 kips per sq. ft.

Ftg. width ft.	Total load kips	Ftg. wt. kips	c in.	d in.	A_s sq. in.	Max. bar size plain	Max. bar size def.
			F'_c = 2,000 psi.				
3.0	72	1	9	7.7	**2.30**		#4
3.5	98	2	9	9.4	**2.55**		4
4.0	128	3	9	11.2	**2.80**		4
4.5	162	4	9	13.0	**3.07**		4
5.0	200	6	10	14.4	3.55		4
5.5	242	7	11	15.9	4.29		4
6.0	288	9	12	17.3	5.11		5
6.5	338	12	13	18.7	5.99		5
7.0	392	15	14	20.2	6.95		5
7.5	450	18	15	21.6	7.98		6
8.0	512	22	16	23.1	9.08		6
8.5	578	25	17	24.5	10.3		7
9.0	648	30	18	25.9	11.5		7
9.5	722	35	19	27.4	12.8		7
10.0	800	41	20	28.8	14.2		8
10.5	882	47	21	30.3	15.6		8
11.0	968	54	22	31.7	17.2		9
11.5	1058	61	23	33.1	18.8		9
12.0	1152	70	24	34.6	20.4		10
12.5	1250	78	26	35.7	22.0		10
13.0	1352	89	27	37.1	23.8		10
13.5	1458	98	28	38.6	25.7		11
14.0	1568	110	29	40.0	27.6		11
14.5	1682	121	30	41.5	29.7	ϕ6	11
15.0	1800	135	31	42.9	31.7	6	11
			F'_c = 3,000 psi.				
3.0	72	1	9	6.9	**2.10**		#4
3.5	98	2	9	8.5	**2.30**		4
4.0	128	3	9	10.1	**2.52**		4
4.5	162	4	9	11.7	3.24		4
5.0	200	5	9	13.3	4.06		5
5.5	242	7	10	14.6	4.91		5
6.0	288	9	11	15.9	5.83		6
6.5	338	11	12	17.2	6.84		6
7.0	392	13	13	18.5	7.92		7
7.5	450	17	14	19.8	9.09		7
8.0	512	20	15	21.1	10.3		8
8.5	578	24	15	22.7	11.8		9
9.0	648	28	16	24.0	13.2		9
9.5	722	34	17	25.3	14.7	ϕ5	10
10.0	800	39	18	26.7	16.3	5	10
10.5	882	46	19	28.0	17.9	5	11
11.0	968	51	20	29.3	19.6	5	11
11.5	1058	58	21	30.6	21.4	6	11
12.0	1152	67	22	31.9	23.3	6	11
12.5	1250	74	23	33.2	25.3	6	11
13.0	1352	82	24	34.5	27.4	6	11
13.5	1458	93	25	35.8	29.5	7	11
14.0	1568	103	25	37.4	31.9	7	11
14.5	1682	113	26	38.7	34.2	7	11
15.0	1800	127	27	40.0	36.6	8	11

Total load kips	Ftg. wt. kips	c in.	d in.	A_s sq. in.	Max. bar size plain	Max. bar size def.	Ftg. width ft.
		F'_c = 2,500 psi.					
72	1	9	7.3	**2.19**		#4	3.0
98	2	9	8.9	**2.41**		4	3.5
128	3	9	10.6	**2.65**		4	4.0
162	4	9	12.3	3.06		4	4.5
200	5	10	13.7	3.77		4	5.0
242	7	11	15.0	4.57		5	5.5
288	9	12	16.4	5.44		5	6.0
338	12	12	18.1	6.46		6	6.5
392	14	13	19.4	7.49		6	7.0
450	18	14	20.8	8.59		7	7.5
512	21	15	22.2	9.77		7	8.0
578	25	16	23.5	11.0		8	8.5
648	29	17	24.9	12.4		8	9.0
722	35	18	26.3	13.8		9	9.5
800	40	19	27.6	15.2		9	10.0
882	45	20	29.0	16.8		10	10.5
968	53	21	30.3	18.4		10	11.0
1058	59	22	31.7	20.1		10	11.5
1152	68	23	33.1	21.9		11	12.0
1250	76	24	34.4	23.8		11	12.5
1352	87	25	35.8	25.7	ϕ6	11	13.0
1458	96	26	37.2	27.7	6	11	13.5
1568	105	27	38.5	29.8	6	11	14.0
1682	118	28	39.9	31.9	6	11	14.5
1800	130	29	41.3	34.2	7	11	15.0
		F'_c = 3,500 psi.					
72	1	9	6.6	**2.01**		#4	3.0
98	2	9	8.1	**2.27**		4	3.5
128	3	9	9.7	2.64		4	4.0
162	4	9	11.2	3.41		4	4.5
200	5	9	12.8	4.27		5	5.0
242	7	10	14.0	5.16		5	5.5
288	9	11	15.2	6.14		6	6.0
338	11	12	16.5	7.20		6	6.5
392	13	13	17.7	8.34		7	7.0
450	16	14	19.0	9.57		7	7.5
512	19	15	20.2	10.9		8	8.0
578	23	15	21.8	12.4	ϕ5	9	8.5
648	27	16	23.0	13.9	5	9	9.0
722	33	17	24.3	15.4	5	10	9.5
800	38	18	25.5	17.1	5	10	10.0
882	44	19	26.8	18.8	6	11	10.5
968	50	20	28.0	20.7	6	11	11.0
1058	56	21	29.2	22.6	6	11	11.5
1152	63	22	30.5	24.6	7	11	12.0
1250	70	23	31.7	26.6	7	11	12.5
1352	80	24	33.0	28.8	7	11	13.0
1458	89	25	34.2	31.0	7	11	13.5
1568	100	25	35.8	33.6	8	11	14.0
1682	110	26	37.0	36.0	8	11	14.5
1800	121	27	38.3	38.5	8	11	15.0

For notes pertaining to this table, see page 135.

Table 25
Unreinforced Rectangular Mass Footings



$(F_{ct} = 0.02\ F'_c + 20\ psi.)$

Self weight of footing included:

$$D = \tfrac{1}{2}\sqrt{\left(\frac{0.261\ x^2}{F_{ct}}\right)^2 + \frac{83.3\ x^2\ p}{F_{ct}}} - \frac{0.261\ x^2}{F_{ct}}\ \text{(in.)}.$$

Self weight of footing ignored:

$$D = \sqrt{\frac{20.83p}{F_{ct}}}\ x = \beta\ x\ \text{(in.)}.$$

Procedure:
Enter Table with known values of F'_c, p and x.
For footing weight included, select value of D directly.
For footing weight ignored, select value of β and compute $D = \beta x$.

Bearing Pressure p (ksf.)	Outstand x (in.) 5	10	15	20	25	30	35	40	45	50	55	60	65	70	β
	Footing Depth D (in.)														
	F'_c = 2,000 psi.														
1	3	6	9	11	14	16	19	21	23	25	27	29	31	32	.59
2	5	9	13	16	20	24	27	31	34	37	40	43	46	49	.83
3	6	10	15	20	25	29	34	38	42	46	50	54	58	62	1.02
4	6	12	18	23	29	34	39	44	49	54	59	64	68	73	1.18
6	8	15	22	29	35	42	48	55	61	67	74	80	86	91	1.44
8	9	17	25	33	41	49	56	64	71	79	86	93	100	107	1.67
10	10	19	28	37	46	54	63	72	80	88	97	105	113	121	1.86
15	12	23	34	45	56	67	78	88	99	109	120	130	140	150	2.28
20	14	27	40	52	65	78	90	102	115	127	139	151	163	175	2.63
30	17	33	48	64	80	95	111	126	141	157	172	186	201	216	3.23
40	19	38	56	74	92	110	128	146	164	181	199	216	234	251	3.73
60*	23	46	68	91	113	135	158	180	202	223	245	267	288	310	4.56
	F'_c = 2,500 psi.														
1	3	6	8	11	13	15	17	20	22	24	25	27	29	31	.55
2	4	8	12	15	19	22	25	29	32	35	38	41	43	46	.84
3	5	10	14	19	23	27	31	35	39	43	47	51	55	58	.94
4	6	11	16	22	27	32	36	41	46	51	55	60	64	68	1.09
6	7	14	20	26	33	39	45	51	57	63	69	74	80	85	1.34
8	8	16	23	31	38	45	52	59	66	73	80	87	93	100	1.54
10	9	18	26	34	42	51	59	67	74	82	90	98	105	112	1.73
15	11	21	32	42	52	62	72	82	92	102	111	121	130	140	2.11
20	13	25	37	49	60	72	84	95	107	118	129	140	151	162	2.44
30	15	30	45	60	74	88	103	117	131	145	159	173	187	201	2.99
40	18	35	52	69	86	102	119	136	152	168	185	201	217	233	3.45
60	22	43	63	84	105	126	146	167	187	207	227	247	267	287	4.23
	F'_c = 3,000 psi.														
1	3	5	8	10	12	14	16	18	20	22	24	26	27	29	.51
2	4	8	11	14	18	21	24	27	30	33	36	38	41	44	.72
3	5	9	13	18	22	26	30	33	37	41	44	48	51	55	.88
4	6	11	15	20	25	30	34	39	43	48	52	56	60	64	1.02
6	7	13	19	25	31	37	42	48	54	59	64	70	75	80	1.25
8	8	15	22	29	36	42	49	56	62	69	75	81	88	94	1.44
10	9	16	24	32	40	47	55	62	70	77	84	92	99	106	1.61
15	10	20	30	39	49	58	68	77	86	95	104	113	122	131	1.98
20	12	23	34	45	57	68	78	89	100	111	121	132	142	152	2.28
30	14	28	42	56	69	83	96	110	123	136	149	162	175	188	2.79
40	17	33	49	64	80	96	111	127	142	158	173	188	204	219	3.23
60	20	40	59	79	98	118	137	156	175	194	213	232	251	269	3.95
	F'_c = 3,500 psi.														
1	3	5	7	10	12	14	16	18	19	21	23	25	26	28	.48
2	4	7	10	14	17	20	23	25	28	31	34	36	39	42	.68
3	5	9	13	17	20	24	28	32	35	39	42	46	49	52	.83
4	5	10	15	19	24	28	32	37	41	45	49	53	57	61	.96
6	6	12	18	23	29	35	40	45	51	56	61	66	71	76	1.18
8	7	14	21	27	34	40	46	53	59	65	71	77	83	89	1.36
10	8	16	23	30	38	45	52	59	66	73	80	87	93	100	1.52
15	10	19	28	37	46	55	64	73	81	90	99	107	116	124	1.86
20	11	22	32	43	53	64	74	84	94	105	115	124	134	144	2.15
30	14	27	40	53	65	78	91	104	116	129	141	153	166	178	2.64
40	16	31	46	61	76	90	105	120	135	149	164	178	192	207	3.04
60	19	38	56	74	93	111	129	147	165	183	201	219	237	254	3.73

Example 31 refers

* Vertical compressive stress in footing may be critical (see AS CA2–1963).